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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 1 MARCH, 1924

A WEIGHING STATION USED IN THE CONNECTICUT HIGHWAY TRANSPORTATION SURVEY

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U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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VOL. 5, NO. 1 MARCH, 1924 H. S. FAIRBANK, Editor

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Traffic on the Boston Post Road near New Haven, Conn. Illustrating the retardation of passenger vehicles by motor truck traffic.

CONNECTICUT HIGHWAY TRANSPORTATION SURVEY.

Conducted by the U. S. Bureau of Public Roads and Connecticut State Highway Commission.

A PRELIMINARY REPORT ON THE FIRST THREE MONTHS OF OPERATION.

By J. GORDON McKAY, Highway Economist, U. S. Bureau of Public Roads.

THE Connecticut Highway Transportation Survey, conducted by the U. S. Bureau of Public Roads in cooperation with the Connecticut State Highway Commission, was begun September 11, 1922, and continued for one year.

The two highway agencies entered into the study—the most intensive and carefully planned census of highway traffic which had been attempted—with the conviction that a large volume of highway transportation facts was a prerequisite to the solution of many of the problems of highway construction and maintenance and highway transportation. They were influenced by no prior beliefs or prejudices, and were committed to an unbiased study of the situation in this one State as an initial experiment to be followed by similar studies on the part of the Federal bureau in other States in which it is hoped to have the cooperation of the State highway departments.

Credit is due to Mr. C. J. Bennett, State highway commissioner of Connecticut during the major part of the period of the survey, and to the personnel of the Connecticut highway commission for their excellent cooperation in obtaining the records.

FACTS DERIVED FROM THE SURVEY SUMMARIZED.

A careful analysis of the results of the survey obtained during the first three months reveals certain definite facts which are set forth in this preliminary report, to be followed as soon as possible with supplementary information in greater detail when the entire year's operation has been analyzed.

The following are the principal facts ascertained in the first three months:

Passenger traffic averages 2.5 persons per vehicle and 45.1 miles per car per trip.

Thirty-five per cent of the passenger movement is primarily for business purposes and 65 per cent for nonbusiness purposes.

Passenger business traffic averages 1.7 passengers per car and the nonbusiness traffic 3 passengers per car with an average trip mileage of 29.7 miles for the business and 55.5 miles for the nonbusiness traffic.

Twenty-three and five-tenths per cent of the passenger mileage in the State is for business purposes and 76.5 per cent is for nonbusiness purposes.

The peak of the highway movement occurs in this State in October and the minimum is reached in February, when motor-truck transportation is 40 per cent and passenger traffic is 70 per cent lower than the October movement.

There is a more rapid decline in motor-truck traffic on secondary highways than on the New York-Springfield route, the main interstate highway through the State.

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The percentage of large motor trucks is smaller on the secondary highways.

Motor-truck traffic reaches the daily minimum at 4 a. m., from which time it increases to a first peak between 10 a. m. and 12 o'clock noon and a second peak between 2 and 4 p. m., decreasing rapidly after 5 p. m.

The night movement of motor trucks consists largely of long-distance traffic in special commodities hauled in trucks of large capacity with heavy net loads.

The percentage of overloads is considerably greater at night than during the daylight hours.

The percentages of ½, 1½, 5, and 5½ ton trucks were slightly greater in 1922 than in 1921 in this State and there was a falling off in the percentages of 2, 2½, 3, 4, and 6½ to 7½ ton trucks.

THE PREVALENCE OF OVERLOADING.

Twenty-nine and six-tenths per cent of the trucks operated over the highways of the State were loaded in excess of their rated capacity, but this percentage represented a considerable improvement over the condition observed in 1921, a fact that is attributed to the enforcement of the traffic laws of the State.

Overloading is a practice in which trucks of all makes and capacities are involved, although the indications are that the ½, 1½, and 5½ to 7½ ton trucks are not overloaded as regularly as the 2 to 5 ton trucks, the small percentage of overloading of the 5½ to 7½ ton trucks being attributed in part to the enforcement of the 25,000-pound maximum gross load limit of the Connecticut law.

Sixty-five and two-tenths per cent of the overloaded vehicles are trucks of 1-ton capacity or less.

There is a distinct relation between the width of truck body and the probability of overloading, the percentage of overloads increasing rapidly with increase in body width.

With the exception of the trucks of the 5½-ton class the average trip mileage for trucks of all capacities is less than 70 miles.

The average trip mileage of pneumatic-tired trucks exceeds that of solid-tired trucks in all capacities except the ½ to 1½ ton group.

The average net load per vehicle is greater in solid-tired trucks of all capacities than in pneumatic-tired vehicles.

The net loads carried on extra-State trucks average greater than the net loads carried by Connecticut trucks.

The use of the Connecticut highway system by foreign trucks, on a ton-mile basis, is greater than the use by Connecticut trucks.

Connecticut trucks, constituting 89 per cent of the total motor-truck traffic, transport 86.5 per cent of the total net tonnage and account for only 65.2 per cent of the total mileage, the latter condition being attributed to the fact that the movement of foreign trucks is generally a long-distance movement, while the average mileage made by Connecticut trucks is lowered by the higher percentage of local hauls.

MOTOR-TRUCK MOVEMENT LARGELY IN SHORT-HAUL ZONES.

Of the tonnage moved by motor truck, 36.9 per cent was hauled from 1 to 9 miles, 30.5 per cent from 10 to 29 miles, and 32.6 per cent 30 miles or more.

Of the motor trucks, 78.6 per cent operate on trips of from 1 to 29 miles and transport 67.4 per cent of the total net tonnage; beyond 29 miles 21.4 per cent of the motor trucks transport 32.6 per cent of the total net tonnage.

One-third of the movement of trucks is an empty movement and two-thirds loaded.

The percentage of loaded vehicles was smallest in the 1 to 9 mile zone, while the percentage loaded remained relatively constant for all hauls from 10 to 70 miles.

Of the total movement of trucks in the State, 62.3 per cent may be classed as regular motor-trucking service providing from one to six or more trips per week.

Of the motor trucks, 80.5 per cent handle 70.5 per cent of the net commodity tonnage in door-to-door delivery; 9.6 per cent handle 14.7 per cent of the tonnage in terminal-to-delivery service, the relatively unimportant balance consisting of terminal-to-terminal and pick-up-to-terminal service.

The 5 to 7½ ton groups of motor trucks carry 34.3 per cent of the total net tonnage, and the ½ to 1½ ton group 26.3 per cent.

Trucks of small capacity lead in the 1 to 9 mile haul, while the percentage of 5 and 7½ ton trucks increases as the haul becomes longer.

Of the tonnage transported over the roads of the State, 81.5 per cent originates in the State.

Of the commodities transported by motor truck in the State, 73.6 per cent may be classified as manufactured articles.

The bulk of motor-truck shipment in Connecticut is of the character of short-haul transportation and does not compete with the business of the railroads.

Of the tonnage, 75 per cent is transported under contract between shippers and truck operators.

The fact that motor-truck rates may be below the level of rail rates constitutes a factor secondary in importance to rapid delivery and trade demands in determining the manufacturer's method of shipment.

The improvement of rail service beyond the 30-mile haul has resulted in a decreased use of motor trucks in the long haul, except for a few specialized commodities.

Actual or potential competition of motor-trucking companies with rail or water services is an incentive to both rail and water operating companies to provide effective transportation.

THE NEED FOR INVESTIGATION OF HIGHWAY TRANSPORTATION.

The foregoing facts are the first conclusions drawn from the survey. The Bureau of Public Roads is convinced that the solution of problems encountered in the highway transportation of freight and passengers depends upon the development of a sufficient volume of similar data in typical sections of the country to provide an authentic basis for analysis, discussion, and an intelligent formation and appraisal of governmental policies. The problems involved are so fundamental and the interests affected so varied and conflicting that impartial and scientific information must be obtained as a basis for a rational solution.

During the past decade the growth of modern State highway systems has increased rapidly, stimulated by the realization of the economic and social values arising from the development of highway systems, the increased economic utility of the motor vehicle, and by the rapid yearly increase of motor-vehicle ownership. The result of this growth is the reemergence of the highway as a factor in the transportation of people and goods.

Trucks of small capacity lead in the 1 to 9 mile haul. Their service does not compete with the railroads. Here are 1½ tons of bananas, and livestock weighing 1½ tons, each a 9-mile load.

The history of the modern highway is so brief and its growth has been so rapid that there is an amazingly meager body of authentic evidence from which we can measure its economic value or determine its economic sphere of operation as a correlated part of our transportation system.

The literature of the past few years dealing with highway transportation, particularly the transportation of freight, contains a variety of conflicting predictions and conclusions, usually based upon fragmentary or localized data rather than facts.

The highway engineer of to-day is limited on the one hand by lack of revenue for construction and maintenance of highways, and hard pressed on the other hand to maintain adequate highway service for the amazing growth of traffic, and he is critical of traffic elements which, from his point of view, are destructive of the highways or increase the original cost of the investment.

CLAIMS OF INTERESTS CONFLICT.

Some of the promotion literature advocates the increased development of highway intercity transportation of freight without basic evidence as to the comparative economies of operation or an economic knowledge as to whether this new transportation factor should be a supplementary or competitive agency in relation to existing transportation systems, and is usually colored by a desire to create or extend markets for the sale of the instruments of highway transportation.

The increase in the costs of highway construction and maintenance, due largely to the increasing volume of traffic, coupled with the difficulty of raising highway revenue to cover these costs, has directed the attention of State legislatures to the highway transportation of freight and passengers, resulting in an increase of regulatory measures affecting highway transportation.

The rapid development of motor-transport organizations engaged in the transportation of freight during the war period of rail congestion was the real beginning of intercity trucking. This recent expansion of highway freight transportation has directed the attention of the railroad operators to this new, so-called competitor. Particular attention is directed by the railroad management to the taxation of the railroad investment, a portion of this revenue being used for the construction of highways paralleling rail lines, which, it is contended by the rail management, provides a right of way for a competitor.

The enthusiastic promotor of intercity transportation of freight by motor truck insists that the motor truck will relieve the rail lines of an uprofitable short haul of L. C. L. (less-than-carload) freight and reduce operating and terminal charges. As a matter of fact the terminal problem is confined to a limited number of cities and is not applicable to the country as a whole. The rail terminal freight costs of New York City are the exception rather than the rule. The elimination of L. C. L. freight transportation in the short-haul rail zone may be desirable, but it will not necessarily reduce the fixed and operating expenses of rail freight stations already in existence as long as industries in the tributary area continue to originate rail C. L. (carload) freight, and passenger, express, and mail service must be maintained.

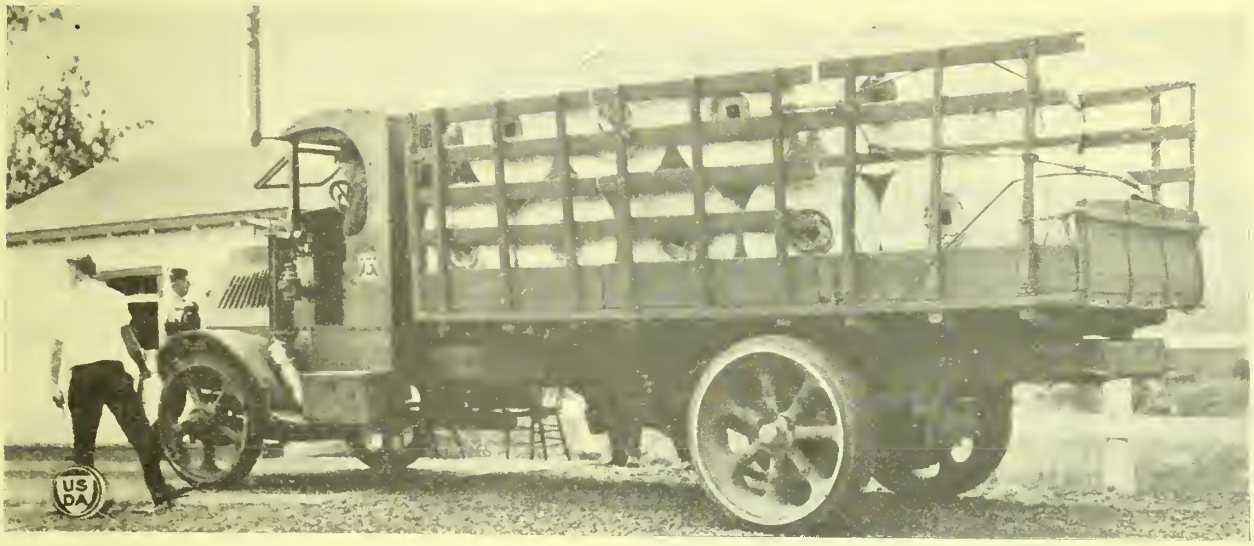
The development of highway transportation has created many difficult problems. The above suggestions are a few of the difficulties, and it is inconceivable that the problems of highway transportation and its relation to other forms of transportation can be answered satisfactorily by theorizing, or that solutions can be accepted as final if based upon individual judgments or fragmentary evidence. The facts of highway transportation in one section of the country are not necessarily applicable to another section unless the industrial and agricultural production as well as the rail and highway facilities are somewhat similar.

The primary purpose of this research project, which was completed in September, 1923, was to assemble all the salient highway facts necessary to a critical analysis of highway transportation in Connecticut, where the transportation of people and commodities is as highly developed, at the present writing, as in any other section of the country.

PURPOSES OF THE CONNECTICUT HIGHWAY TRANSPORTATION SURVEY.

The specific purposes which it was hoped to serve by the Connecticut survey were as follows:

1. To serve as an aid in allocating construction and maintenance funds according to the distribution of traffic over the highway system.
2. To predict future traffic.
3. To determine the daily and seasonal traffic density of passenger cars and motor trucks on primary and secondary highways.
4. To determine the seasonal variations in the traffic movement of vehicles, commodities, net and gross loads for trucks of various capacities, commodity and capacity mileage.
5. To measure gross truck loads, net commodity loads, overloads above the capacity of the vehicles, overloads per inch of tire width on the pavement under load, and overloads per inch of tire width channel to channel.
6. To determine the average clearance width of trucks by capacities, and the relation of the width of truck body to truck overloading.
7. To determine the net tonnage movement of freight transported by regular and irregular trucking operators, the net tonnage of freight transported between cities, and the net freight tonnage of products of agriculture, products of animals, products of mines, products of forests, and manufactured products transported between points of origin and destination.
8. To determine the average length of haul of motor-truck shipments by commodities, capacities, and production areas.
9. To determine the type of freight shipment by motor truck—whether a pick-up-and-delivery or terminal-to-terminal movement, and the packing and crating requirements for classes of commodities shipped. The results will indicate for the same commodities any difference in the packing and crating of freight for truck and rail shipment, and the prevailing type of shipment, pick-up-and-delivery or terminal-to-terminal.
10. To gather data concerning the relation of motor transport to other methods of transportation, particularly as to rates, commodity freight classification, schedules of operation, delivery time between points, average commodity mileage, net tonnage, rates, and delivery time in the short, middle-distance, and long-haul zones.
11. To determine the average number of passengers and mileage per vehicle and the percentage of business and nonbusiness movement of passenger cars.
12. To summarize the yearly passenger and commodity traffic over a State highway system, this summarization to serve as the basis of determining the service value of the State highway system for the transportation of passengers and freight.



Manufactured articles form the bulk of the commodities transported by truck in Connecticut. These typical loads consist of 6¾ tons of wire, hauled 26 miles; 6½ tons of typewriters, hauled 113 miles; and 4½ tons of cardboard, hauled 18 miles.

METHOD OF CONDUCTING THE SURVEY.

The successful operation of the survey required careful planning of the data to be recorded, well-worked-out field operating schedules to insure accurate data and avoid duplicate recording of traffic, and, finally, a systematic analysis of the information.

An intensive weigbing station. Observers weighing and measuring a 4-ton load of rabbit fur, bound from New York City to Norwalk, Conn., 44 miles.

The recording of the field data is not as difficult as the statistical analysis of the information. Empty-weight standards for all trucks have been evolved, based upon actual empty weights for each make and capacity of truck, with an allowance for body weight. It was necessary to set up these standards before it was possible to determine the net tonnage of loaded vehicles. Commodity codes were required, including all of the variety of commodities transported in the area, to facilitate the rapid analysis of commodity tonnage and movement. Mileage charts between origins and destinations in the area observed had to be computed in advance to simplify the reduction of mileage data. Rules and regulations governing the recording of data and the coding of information were carefully formed to insure the successful handling of the volume of records. Night surveys were made at frequent intervals to enable a correction of the daily records to a 24-hour basis.

THE SURVEY—A MODERN DEVELOPMENT IN HIGHWAY RESEARCH.

The Connecticut Transportation Survey of 1922 is a modern development in highway research. Highway transportation surveys usually record for a brief period the number and type of vehicles operating over a highway system. The experience of the U. S. Bureau of Public Roads in conducting the California Highway Transportation Survey, in 1920, the Tennessee Highway Transportation Survey of 1921–22, and the Connecticut Highway Transportation Survey of 1921 was valuable in planning the Connecticut Highway Transportation Survey of 1922. It can be called a pioneer attempt to determine fundamental facts and is perhaps a standard method of measuring highway transportation. The method will be improved as more researches of a similar nature are undertaken, yet at the present time it is producing essential facts necessary to a critical analysis of highway transportation.

The survey includes two types of researches (fig. 1).

(1) Intensive data, including detailed motor-truck information, were recorded at eight key stations.

(2) Extensive data were recorded at 56 stations, divided into eight districts, with one intensive weighing station as the nucleus of the extensive recording stations in each district.

The combination of intensive and extensive data insures a reasonably accurate measure of highway transportation.

Weight scales were operated at the eight intensive stations one week every two months The extensive recording stations were operated one day each month. By moving from station to station on a regular schedule of operation seasonal variations of traffic were observed.

THE INFORMATION RECORDED.

The following is a brief résumé of the principal types of information recorded at the 8 intensive and the 56 extensive traffic stations.

Intensive traffic stations.—Density of traffic; motor-truck makes; capacity; trips per week; trip time; origin; destination; mileage; commodity; type of shipment, whether pick-up-and-delivery or terminal-to-terminal; net and gross weights; State of license; packing and crating of commodities; tire data; regular and irregular motor-truck operators.

Extensive traffic stations—For passenger vehicles.—Density; State of license; passengers per vehicle; business or nonbusiness usage; origin; destination and mileage.

For motor trucks.—Density; State of license; make; capacity; commodity; trips per week; trip time; origin; destination and mileage.

The data recorded in the field at the intensive and extensive stations were forwarded to the U. S. Bureau of Public Roads at Washington and after the material

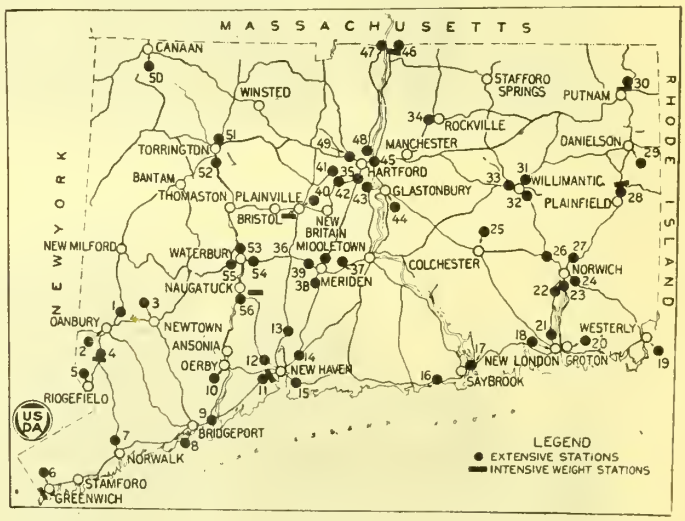


FIG. 1.—Location of extensive and intensive stations in the Connecticut Highway Transportation Survey.

was coded and organized it was punched on tabulating cards. It would be impossible to analyze successfully the enormous volume of traffic records without making use of mechanical sorting and tabulating machines.

DENSITY, SEASONAL VARIATION, AND COMPOSITION OF TRAFFIC.

The following facts are derived from an analysis of the records of 40,613 motor trucks, 16,549 intensive and 24,064 extensive, recorded from September 11 to December 3, 1922, and 175,346 passenger-car records recorded from September 11, 1922, to February 11, 1923.

Daily traffic density and distribution of motor vehicles are illustrated by Figures 2, 3, and 4. These figures show clearly:

1. That the primary highway system in Connecticut carries the bulk of the highway traffic.
2. The major and minor highways and the traffic routes which require the largest outlay for construction and maintenance.

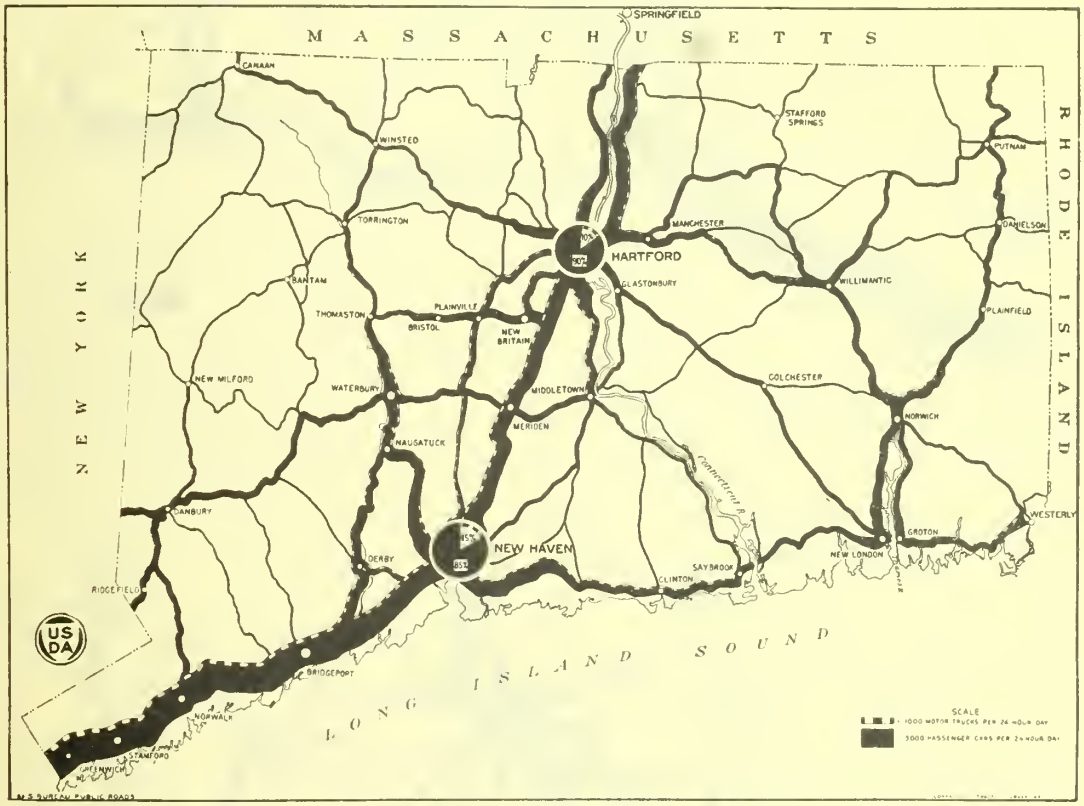


FIG. 2.—Average daily movement of motor trucks and passenger cars on the Connecticut highway system.

3. The major traffic routes which require constant supervision of construction, maintenance, and policing to insure service and safety to traffic.
4. That the greatest density of highway traffic in Connecticut is between large centers of population, production, and distribution.
5. That motor-truck traffic forms a considerably larger proportion of the total traffic on the Boston post road from New Haven, Conn., to New York City, Hartford, and Springfield, Mass., than on the less heavily traveled highways.
6. That at New Haven 15 per cent of the total traffic is made up of motor trucks and 85 per cent of passenger cars, and that at Hartford 10 per cent is made up of motor trucks and 90 per cent of passenger cars.

The percentage of trucks of larger capacity and the average net and gross loads per vehicle on secondary highways are considerably less than on primary highways. The influence of New Haven, Hartford, Bridgeport, and New York City as primary centers of distribution of commodities is indicated by the motor-truck density contiguous to these centers. The type of production in an area influences the volume of motor-truck traffic.

All types of passenger traffic average 2.5 persons per vehicle and 45.1 miles per car per trip. Of this passenger-vehicle movement 35 per cent was for busivess purposes and 65 per cent for nonbusiness purposes. Passengers average 1.7 per car for business usage and 3 for nonbusiness usage, with an average mileage per trip of 29.7 miles for business and 55.5 miles for nonbusiness travel. The real test of business and nonbusiness utilization of a highway system by passenger vehicles is indicated by the fact that 23.5 per cent of the passenger mileage is for business purposes with 76.5 per cent for nonbusiness usage. The passenger usage of the Connecticut highway system is largely a nonbusiness, noncompetitive usage.

OCTOBER—THE MONTH OF MAXIMUM TRAFFIC.

Connecticut motor-truck and passenger traffic varies in volume according to the season as illustrated by Figures 5 and 6. The traffic during the winter months operated under favorable conditions due to the Connecticut highway department's policy of keeping the highways free of snow. Beginning in November a steady decrease in traffic is apparent, reaching the minimum of decline in February, when motor-truck transportation was 40 per cent and passenger-car traffic 70 per cent lower than the October movement. The decline in motor-truck net tonnage during this

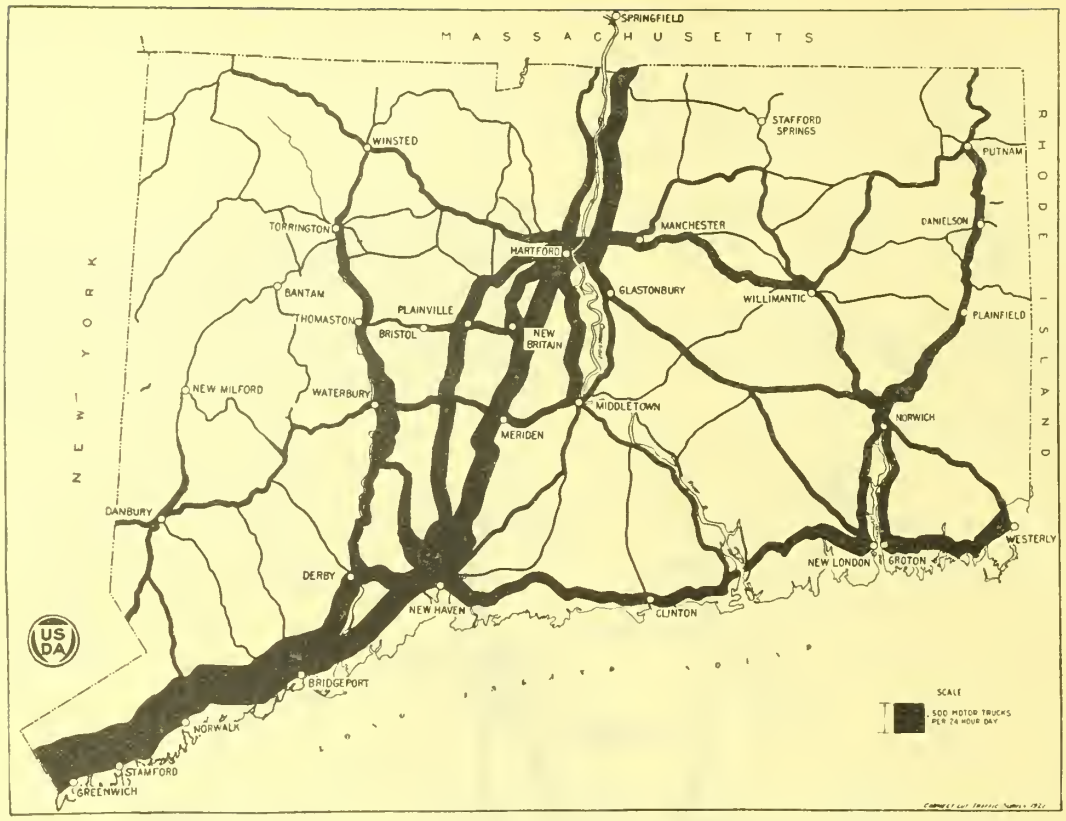


FIG. 3.—Average daily movement of motor trucks on the Connecticut highway system. Width of lines indicates density of traffic.

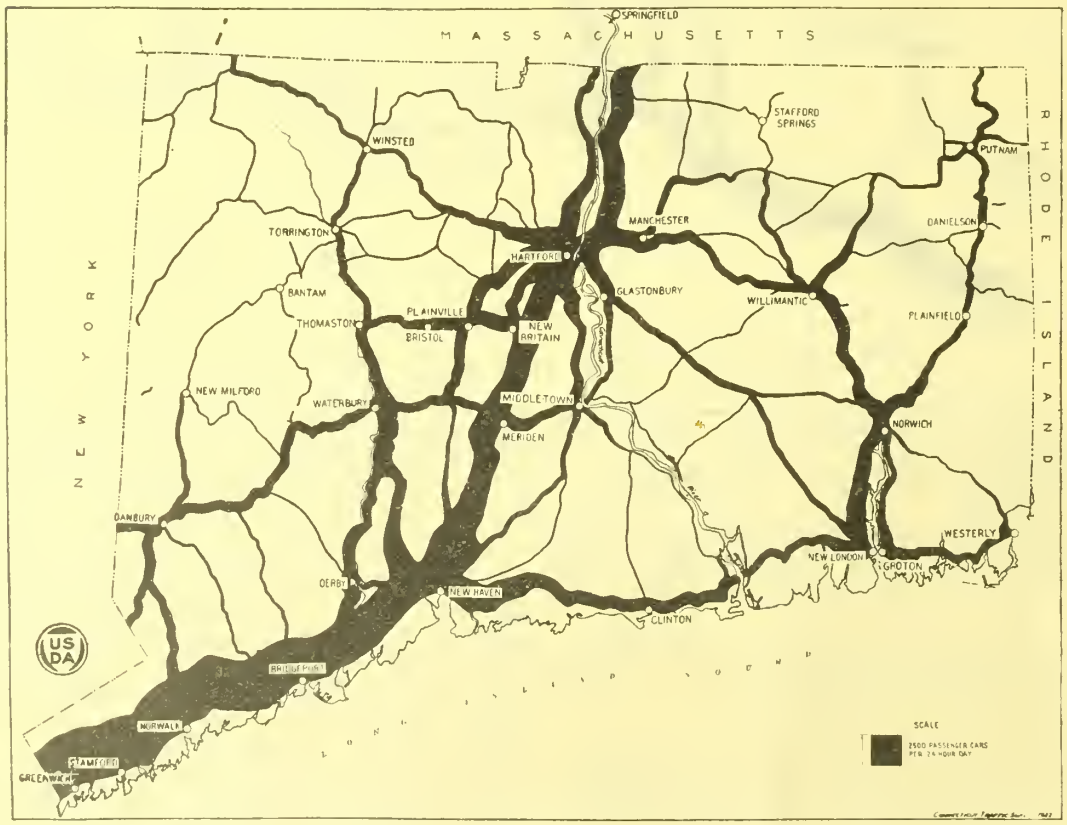


FIG. 4.—Average daily movement of passenger cars on the Connecticut highway system. Width of lines indicates density of traffic.

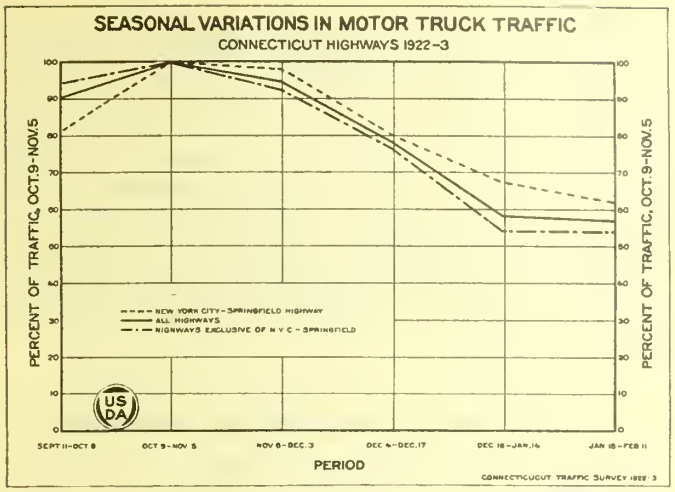

SEASONAL VARIATIONS IN MOTOR TRUCK TRAFFIC
CONNECTICUT HIGHWAYS 1922-3
PERCENT OF TRAFFIC, OCT. 9 - NOV. 5
NEW YORK CITY - SPRINGFIELD HIGHWAY
ALL HIGHWAYS
HIGHWAYS EXCLUSIVE OF N Y C - SPRINGFIELD
SEPT 11-OCT 8
OCT 9-NOV 5
NOV 6-DEC 3
DEC 4-DEC 17
DEC 18-JAN 14
JAN 15-FEB 11
PERIOD
CONNECTICUT TRAFFIC SURVEY 1922-3

FIG. 5.—Motor-truck transportation in February was 40 per cent lower than in October.

period is partly explained by a decrease in the value of perishable and seasonal commodities shipped by truck during the winter months. The improvement in rail service in this area during the same period resulted in a shifting of a certain percentage of the tonnage from motor-truck back to rail transportation. Motor-truck transportation on secondary highways decreased more rapidly than on the New York City-Springfield, Mass., route, the main interstate highway through Connecticut.

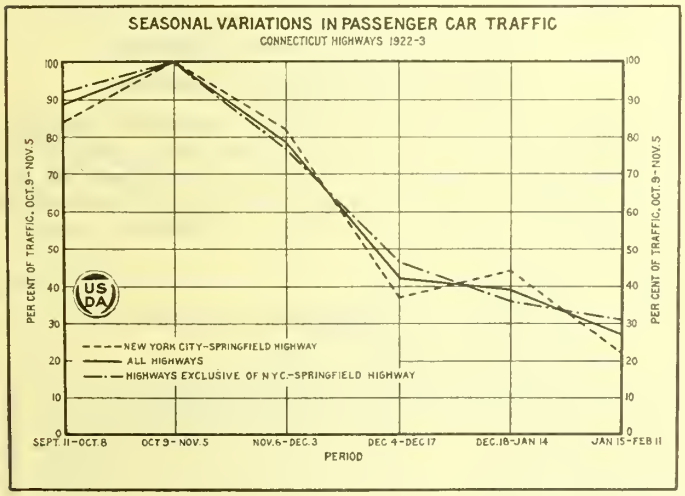


FIG. 6.—The movement of passenger cars was 70 per cent lower in February than in October.

The percentage of trucks of large capacity is smaller on the secondary than on the primary highway system. The decline of motor-truck transportation during the winter months indicates a lack of reliable service which is one of the essentials to the stability of any type of transportation.

Motor-truck traffic, as indicated in Figure 7, begins to increase in volume at 4 a. m., reaching the first traffic peak between 10 a. m. and 12 noon, and the second traffic peak between 2 p. m. and 4 p. m. Motor-truck traffic decreases rapidly after 5 p. m. The night movement from 8 p. m. to 4 a. m. is largely that of the long-distance truck operators transporting special commodities in large-capacity trucks with heavy net loads. The percentage of overloads is considerably larger at night than during the daylight hours.

Comparison of the composition of truck traffic in 1921 and 1922, Figure 8, indicates that the capacity distribution has not changed materially, there being a slight increase in 1922 of $\frac{1}{2}$ to $1\frac{1}{2}$ ton and 5 to $5\frac{1}{2}$ ton capacities and a decrease in the 2, $2\frac{1}{2}$, 3, 4, and $6\frac{1}{2}$ to $7\frac{1}{2}$ ton capacities. Figure 9 shows the prevailing use of the smaller capacities as indicated by the extensive survey, which is representative of traffic on all the highways

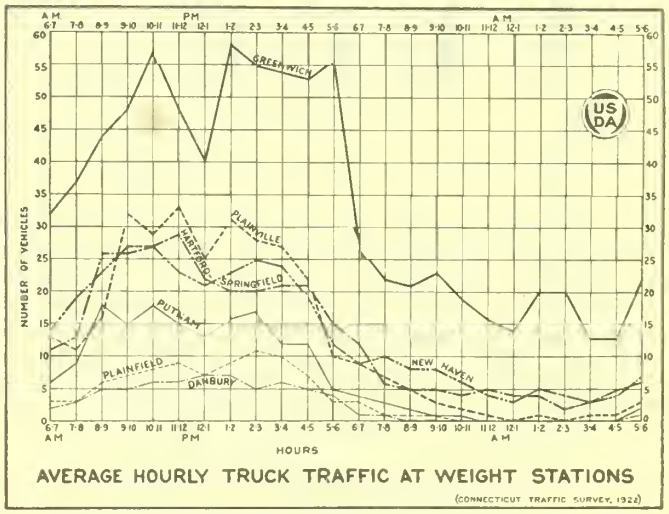


FIG. 7.—Motor-truck traffic is heaviest from 10 a. m. to 6 p. m. The night movement is largely a long-distance movement.

in Connecticut. Both extensive and intensive records indicate the smaller percentage of $6\frac{1}{2}$ to $7\frac{1}{2}$ ton motor trucks operating on the highways.

MOTOR TRUCK OVERLOADING.

Analysis of loaded motor trucks shows that 29.6 per cent are loaded above rated capacity. Motor truck overloading is an uneconomical practice and forces the State to police its highway system constantly to prevent

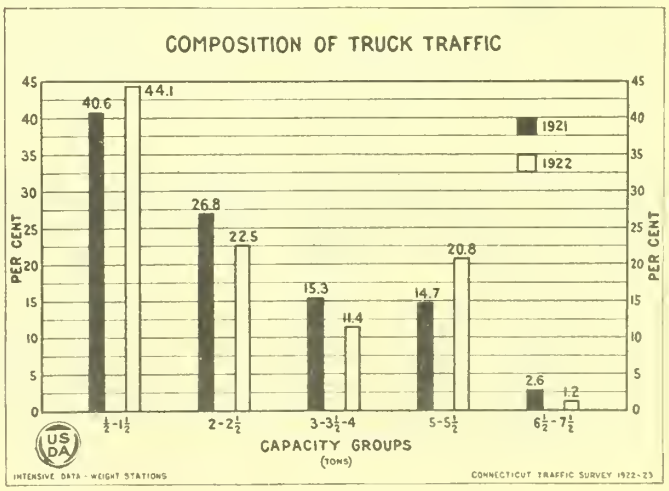


FIG. 8.—Composition of truck traffic in 1921 and 1922.

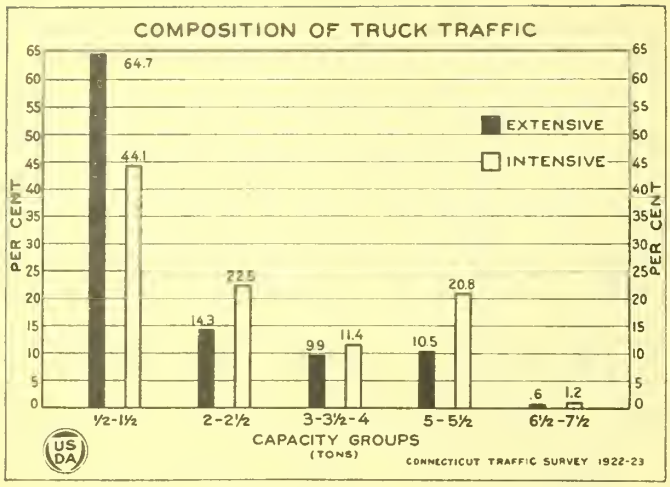


FIG. 9.—Composition of motor-truck traffic by capacity groups.

it. The influence of highway-law enforcement in Connecticut can be seen in Figure 10. The percentage of overloading per capacity in 1922 was considerably lower than in 1921. This decrease was partly due to a realization by motor-truck owners of the rapid depreciation of their vehicles when constantly overloaded, but primarily it was due to police enforcement of the Connecticut laws regulating truck loads.

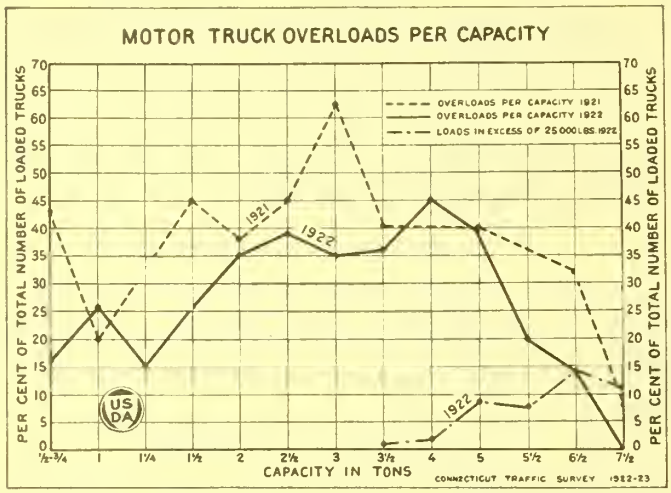


FIG. 10.—The effect of law enforcement is shown by the difference in overloading between 1921 and 1922.

Frequent police inspection and weighing of loaded motor vehicles suspected of being loaded above capacity tends to decrease the practice, especially if the penalty for overloading is sufficient to discourage a repetition of the offense.

Overloading is practiced with all makes and capacities, although the ½ to 1½ ton and 5½ to 7½ ton trucks are not overloaded as regularly as the 2 to 5 ton group. The small percentage of overloads of 6½ to 7½ ton vehicles is partly due to the 25,000-pound maximum gross load limit in Connecticut.

The amount and extent of overloading practiced on Connecticut highways is further illustrated in Figures 11 and 12. Of the total number of overloads 65.2 per cent are 1 ton or less and 34.8 per cent range from 1 to 5 tons per vehicle. Figure 13 clearly indicates that for all capacities an increase in truck body width results in a rapid increase in the percentage of overloads per capacity. Reasonable maximum truck body width and length standards will reduce the practice of overloading. Efficiently organized motor-trucking companies realize the damage to truck equipment when overloaded and regulate their loads to the safe carrying capacity of the trucks.

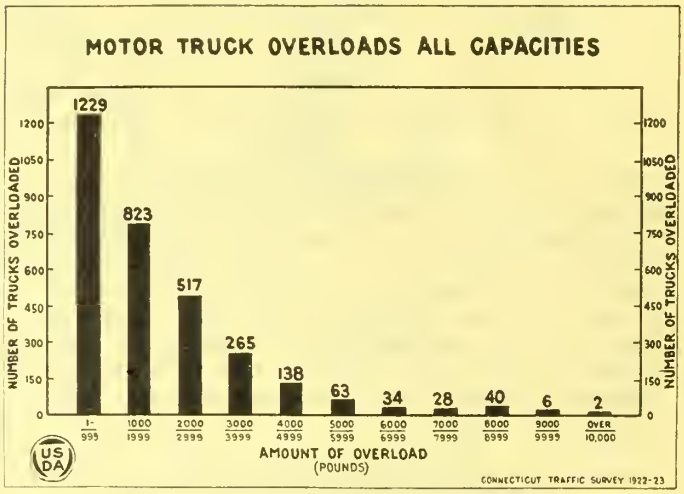


FIG. 11.—The amount and extent of overloading.

MOTOR TRUCK NET WEIGHT AND MILEAGE.

Figure 14 indicates that with an increase in capacity there is an increase in the net load and a rise in the average truck mileage. With the exception of the 5½-ton trucks the average trip mileage of trucks of all capacities is less than 70 miles, indicating that the

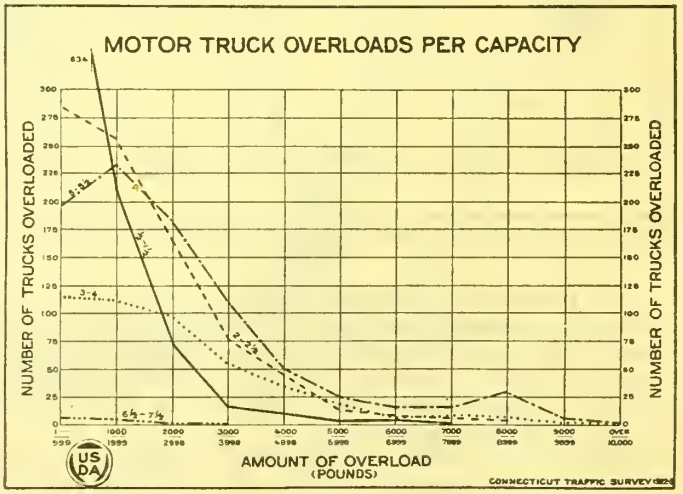


FIG. 12.—The small percentage of overloading of 6½ to 7½ ton vehicles is partly due to the 25,000-pound maximum load limit.

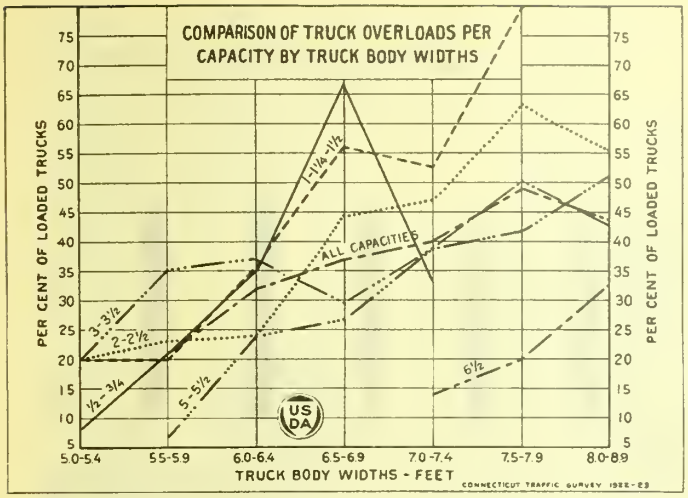


FIG. 13.—The percentage of overloading increases rapidly with increase in body width.

zone of truck transportation, according to the every-day practice of the majority of operators of trucks of all capacities, is less than 70 miles.

The average mileage of pneumatic-tired trucks exceeds the average mileage of solid-tired trucks of all capacities except the ½ to 1½ ton group. (See fig. 15.) The 5 to 7½ ton pneumatic-tired truck mileage is 107.8 miles, while that of the solid-tired trucks of the same capacity group is 61.4 miles. Although the pneumatic-tired truck mileage exceeds the solid-

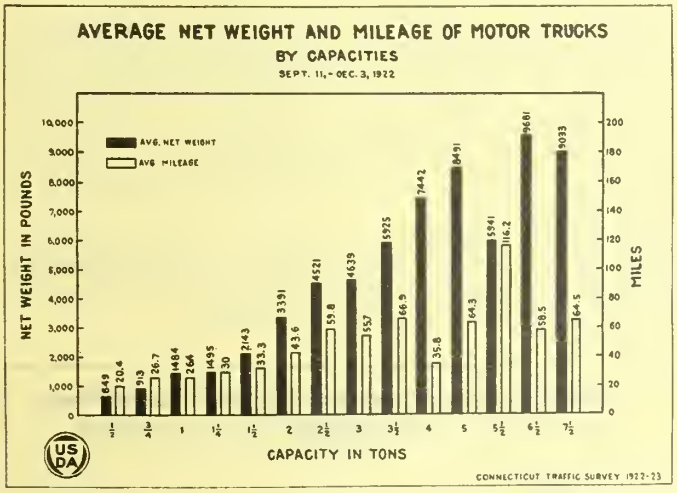


FIG. 14.—Net loads and daily mileage increase with truck capacity.

tired truck mileage, except in the ½ to 1½ ton group, the average net load per vehicle of solid-tired trucks in each capacity class exceeds the average net load of pneumatic-tired vehicles. This is explained by the fact that the present tendency is to use pneumatic-tired trucks for rapid delivery over long distances, transporting commodities of relatively high value and low weight. This type of transportation service is economically justifiable and will remain a permanent part of our transportation system.

Figure 16 illustrates the difference in average mileage of motor trucks from the several States operating on the Connecticut highway system. The average mileage of foreign trucks exceeds the average mileage of Connecticut trucks, partly because they have traveled a considerable mileage before reaching the Connecticut highways. Two tendencies, however, are indicated:

1. The average net loads of foreign trucks exceed those of Connecticut trucks, due largely to the fact that foreign trucks operating over the Connecticut highway system are heavily loaded and of larger capacities.
2. On a ton-mile basis foreign trucks use the Connecticut highway system more per vehicle than Connecticut trucks.

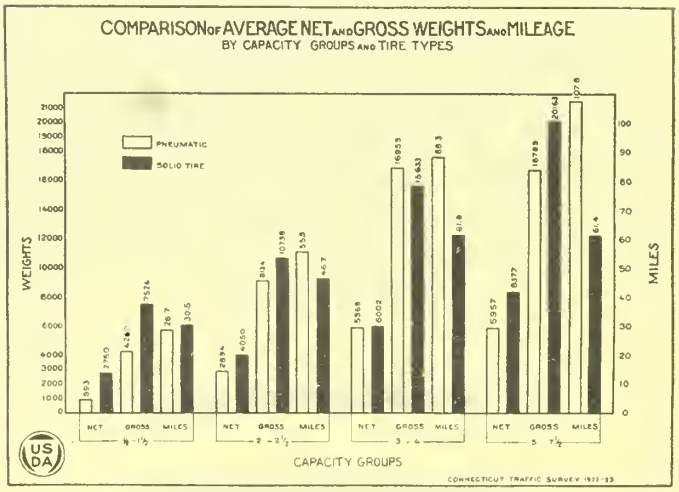


FIG. 15.—Net loads of solid-tired trucks exceed those of pneumatic-tired vehicles, but average mileage of the latter is greater.

CONNECTICUT AND FOREIGN TRUCKS, NET TONS AND MILEAGE.

Figure 17 illustrates the fact that 89 per cent of the motor trucks on the Connecticut highway system are Connecticut trucks transporting 86.5 per cent of the total net tonnage, and 11 per cent are foreign trucks transporting 13.5 per cent of the total net tonnage. Connecticut trucks, 89 per cent of the total vehicles,

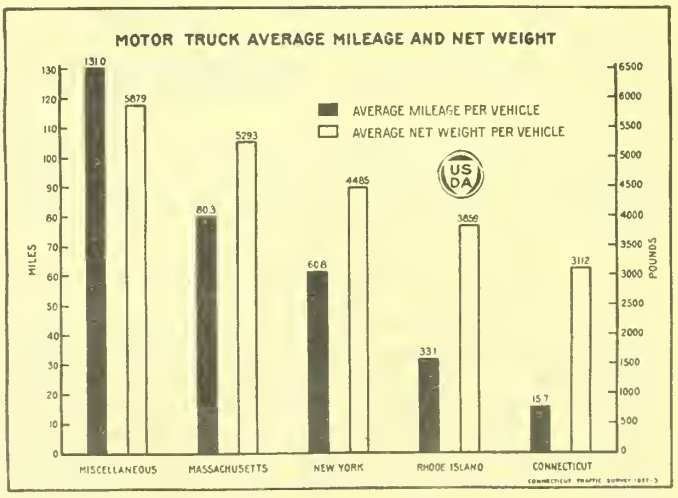


FIG. 16.—Mileage and net weight of trucks from various States.

transport commodities 65.2 per cent of the mileage, while foreign trucks, 11 per cent of the vehicles, transport commodities 34.8 per cent of the mileage.

The explanation of the high percentage of total mileage of foreign trucks is as follows: Mileage figures in Figure 17 are based on origin to destination and include the mileage on all highways—In Connecticut, Massachusetts, New York, Rhode Island, etc. We would expect the Connecticut truck mileage utilization of foreign highways to offset to some extent the foreign truck mileage utilization of Connecticut highways. The foreign truck movement is, as a general rule, a longer-distance traffic, while the Connecticut mileage is lowered considerably by the influence of the Connecticut local hauls.

This analysis, supplemented by a comparison of Connecticut and foreign truck ton-mileage exclusively on the Connecticut highway system, indicates the fact that foreign trucks make a larger ton-mile use of the Connecticut highways than Connecticut trucks.

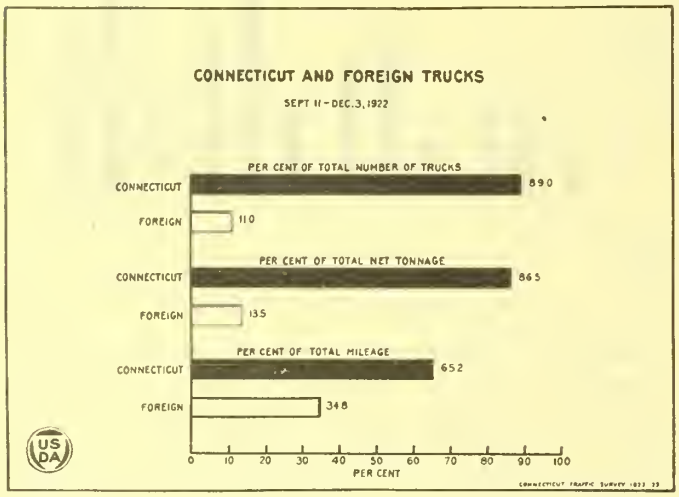


FIG. 17.—Comparison of Connecticut and foreign trucks on basis of number, tonnage, and mileage.

The highways of Connecticut are the logical New England gateways for highway transportation, and naturally foreign trucks use the Connecticut highway system in terms of ton-miles more than do Connecticut trucks.

The practice of operating trucks in Connecticut and licensing the truck in an adjacent State to avoid the higher Connecticut license fee increases the foreign truck usage of the Connecticut highways.

RETURN LOADS AND TRIPS PER WEEK.

One of the difficulties in motor-truck transportation is the problem of return loads. The successful motor-truck operator competing for business must be able to obtain return cargoes or profits decrease. The net rate of return in the motor-truck transportation of freight depends, to a considerable extent, upon the ratio of empty to loaded movement. When the number of motor-truck companies or the number of units offering highway transportation service between two or more points exceeds the needs of shippers of freight the percentage of empty and partially loaded vehicles rises

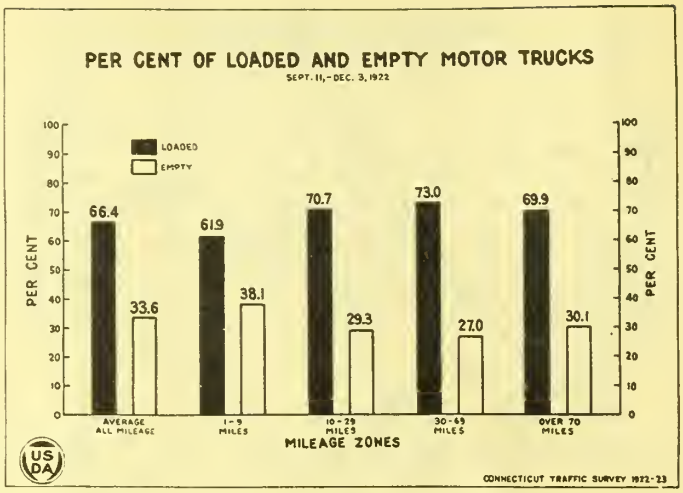


FIG. 18.—Two-thirds of all trucks observed were loaded.

and net profits decrease. The higher the percentage of empty movement the lower will be the rate of return and the keener the competition between truck operators. Figure 18 shows that two-thirds of all motor trucks were loaded, one-third empty. The percentage of loaded vehicles is smallest in the 1 to 9 mile zone, due largely to local distribution, while in the longer hauls, from 10 to 70 miles and over, the percentage of loaded vehicles is approximately the same, ranging from 69.9 per cent to 73 per cent, indicating that the empty factor is relatively constant in both long and short hauls.

The type of production in an area influences the ability of motor truckers to obtain return loads. When manufactured products are shipped by truck and raw materials are shipped in by rail, motor-truck return loads are largely restricted to consumption articles. An extension of the practice of pooling return-load business will increase return loads of motor-truck operators and increase profits.

Figure 19 indicates the number of trips made per week by motor-truck operators and provides a basis for judging the degree of stability of motor-truck service in the highway transportation of freight.

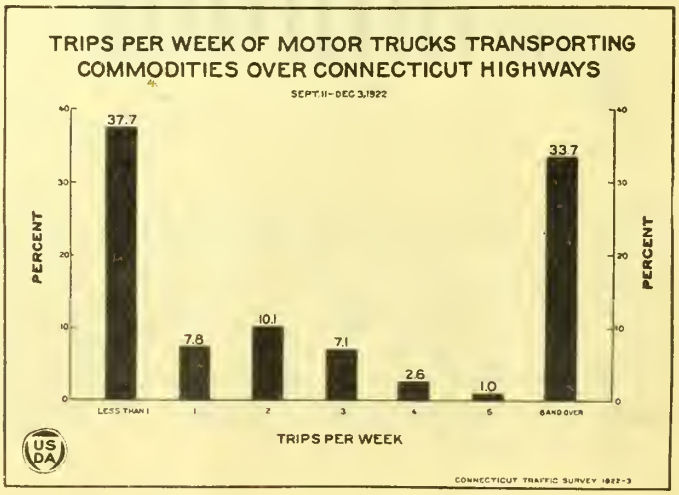


FIG. 19.—The number of trips per week is an indication of the stability of motor-truck service.

Almost two-thirds of the motor-truck movement may be classed as regular trucking service, providing from one to six or more trips each week.

It is interesting to find that 62.3 per cent of the movement can be classed as regular motor-trucking service providing from one to six or more trips per week, while 37.7 per cent is by irregular or intermittent operators making less than one trip weekly.

THE TYPE OF SHIPMENT.

One of the advantages of rural transportation of freight by motor truck is the convenience of pick-up-to-delivery service. Figure 20 shows that 80.5 per cent of the motor trucks handle 70.5 per cent of the net commodity tonnage in door-to-door delivery. Motor-truck transportation from terminal to delivery ranks second in importance, 9.6 per cent of the motor trucks handling 14.7 per cent of the net commodity tonnage. Terminal-to-terminal and pick-up-to-terminal motor-truck freightage are the least important.

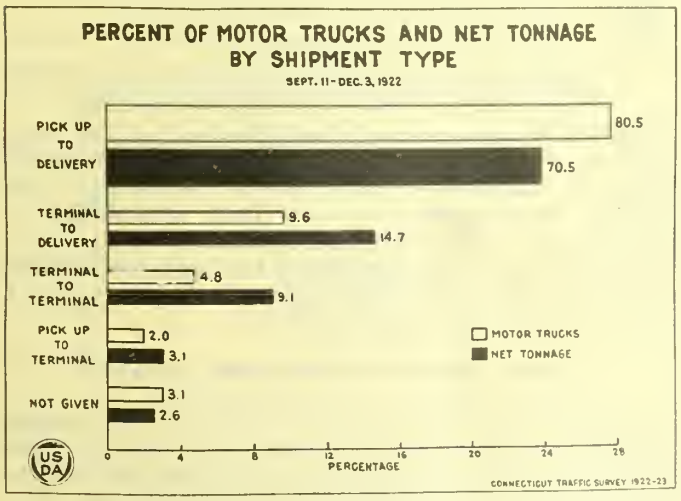


FIG. 20.—Convenient pick-up-to-delivery service is one of the advantages of motor-truck transportation.

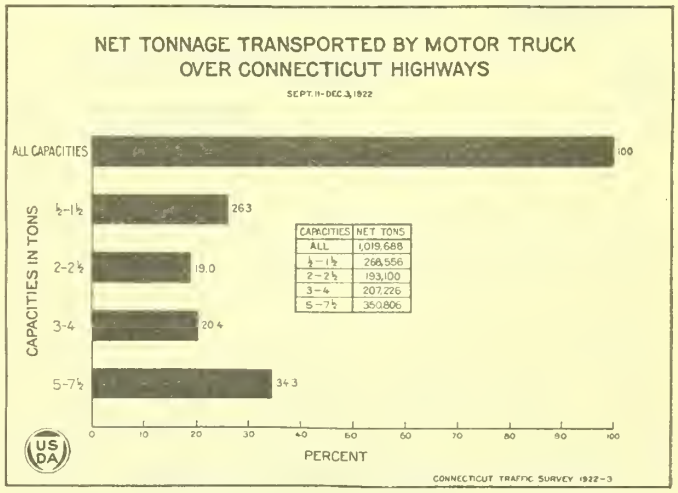


FIG. 21.—Percentage of total net tonnage carried by trucks of several capacity groups.

It can not be said that any particular truck capacity is especially adapted to the motor-truck transportation of freight. Figure 21 indicates that the percentage of net tonnage by motor-truck capacity groups does not vary widely. The 5 to 7½ ton class ranks first in importance, carrying 34.3 per cent of the total net tonnage, and the ½ to 1½ ton group second, with 26.3 per cent of the total net tonnage. The 5 to 7½ ton trucks are used more in the long haul and the ½ to 1½ ton trucks are used more in the local and short haul.

VOLUME OF FREIGHT MOVEMENT BY HIGHWAY.

During the three months from September 11 to December 3, 1922 (tonnage basis 24-hour day), 1,019,688 net tons of commodities were transported over the Connecticut highway system. (Fig. 22.) The competition of motor-truck transportation with rail transportation of freight is not as serious, however, as one would at first believe when told that motor trucks transported 1,019,688 net tons in Connecticut during a three-month period. The condition of railroad transportation in Connecticut was one of the elements responsible for an increase in motor-truck transportation during this period. The lowered efficiency of rail service due to the rail strike beginning July 1, 1922, naturally resulted in an increase in the highway transportation of freight.

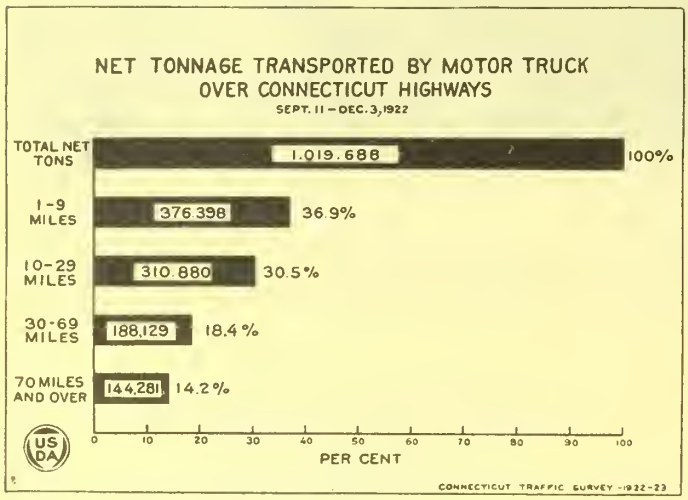


FIG. 22.—Motor-truck traffic in Connecticut divides approximately by thirds into short, middle-distance, and long hauls on the basis of tonnage.

We find that 36.9 per cent of this tonnage was moved from 1 to 9 miles, a local distribution of goods which is noncompetitive. Of this total net tonnage 30.5 per cent was transported from 10 to 29 miles. In this, which may be called the short-haul zone, the tonnage transported is largely noncompetitive. A portion is local distribution from jobbers and wholesalers to retailers. Still another portion is shipped from or to points without rail facilities and by its nature can not be competitive. The balance of the total net tonnage, 32.6 per cent, was transported 30 miles or more. This movement is partially competitive. It is, however, largely a transportation of goods which is not determined by the rate charged for the transportation. Two factors are, in general, responsible for the transportation of these commodities over 29 miles: (1) Lack of efficient rail service enabling shippers to obtain rapid rail transportation of L. C. L. freight; (2) certain types of commodities, such as household goods, wholesale grocery products, meat products, and vegetables, which are especially adapted to motor-truck shipment will probably continue to be shipped by truck for distances beyond the short-haul zone. The total tonnage of these commodities is not very significant.

Figure 23 indicates that 78.6 per cent of the motor trucks are operating from 1 to 29 miles, transporting 67.4 per cent of the total net tonnage. Beyond 29 miles 21.4 per cent of the motor trucks transport 32.6 per cent of the total net tonnage.

Figure 24 shows the relative use of different motor-truck capacities in the local, short, middle-distance, and long-haul transportation of freight by highway.

The small capacities, ½ to 1½ tons, lead in the local 1 to 9 mile zone, decreasing in use with an increase in mileage. The 2 to 2½ and 3 to 4 ton capacities transport approximately the same proportion of tonnage in each of the four mileage zones. The proportion of net tonnage transported by the 5 to 7½ ton capacities increases with mileage from approximately 25 per cent in the 1 to 9 mile zone to 50 per cent in the haulage over 69 miles.

Figure 25 indicates the percentage of motor trucks of various capacities in each mileage zone. The percentage of ½ to 1½ ton capacities decreases with increase in mileage, while the percentage of 2 to 2½, 3 to 4, and 5 to 7½ ton capacities, with but a slight deviation, increases with increased mileage. The large capacities are more generally used in the longer hauls. This is shown by the fact that 66.2 per cent of the motor vehicles transporting commodities from 1 to 9 miles have capacities of ½ to 1½ tons and 7 per cent have 5 to 7½ ton capacities, while 17.4 per cent of the motor vehicles transporting commodities 70 miles or over have ½ to 1½ ton capacities and 38.6 per cent have 5 to 7½ ton capacities.

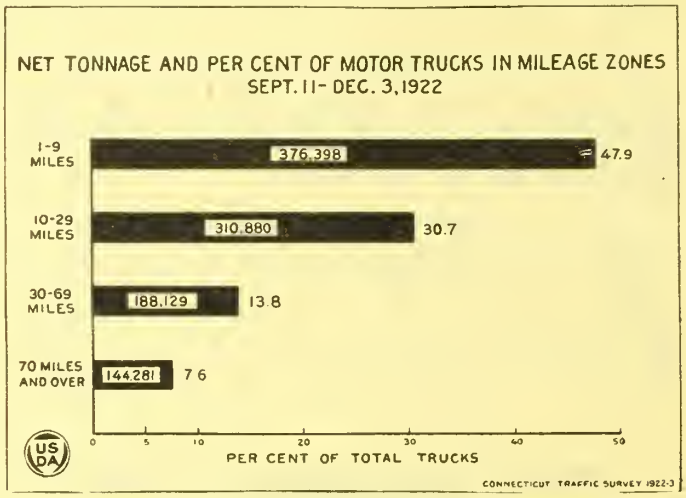


FIG. 23.—The smaller trucks ply in the short and middle-distance zones; the larger trucks take the long haul.

Figure 26 shows clearly that the bulk of the commodities (81.5 per cent) transported over Connecticut highways originates in the State of Connecticut, and 18.5 per cent originates in New York, Massachusetts, Rhode Island, and miscellaneous States.

On the whole the majority of the highway services of the Connecticut highway system are for Connecticut producers and consumers.

BULK OF TRAFFIC IN MANUFACTURED ARTICLES.

Motor-truck transportation of goods in New England is largely limited to the movement of finished or semi-finished manufactured products as illustrated in Figures 27 and 28. It is natural in an industrial area that a larger proportion of the goods transported by motor

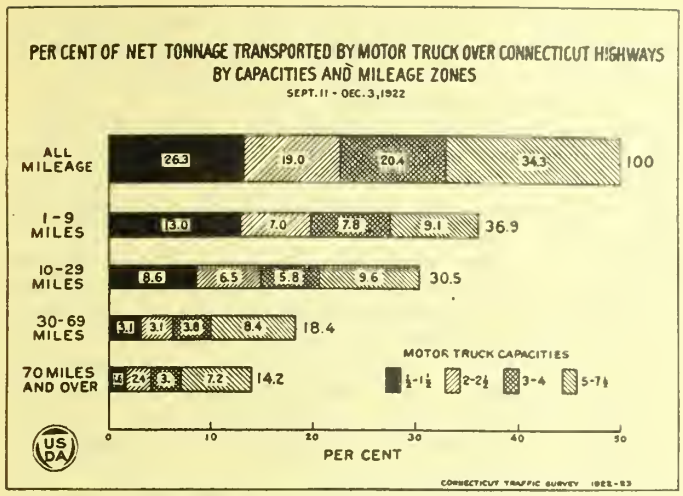


FIG. 24.—Percentage of tonnage carried by trucks of various capacities in several mileage zones.

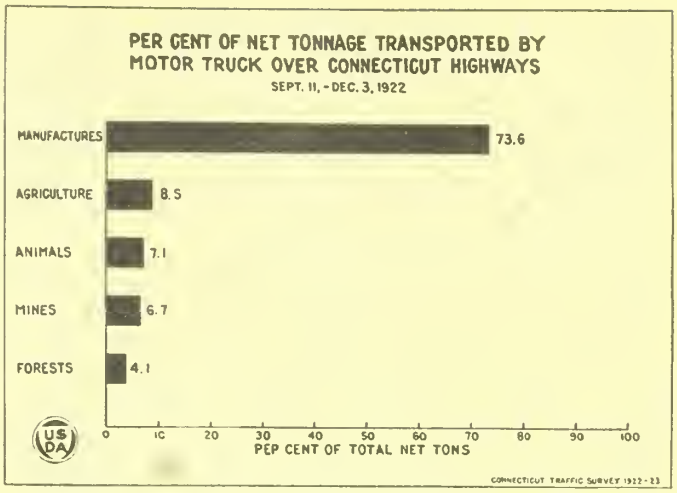


FIG. 27.—Manufactured commodities are transported in greatest volume.

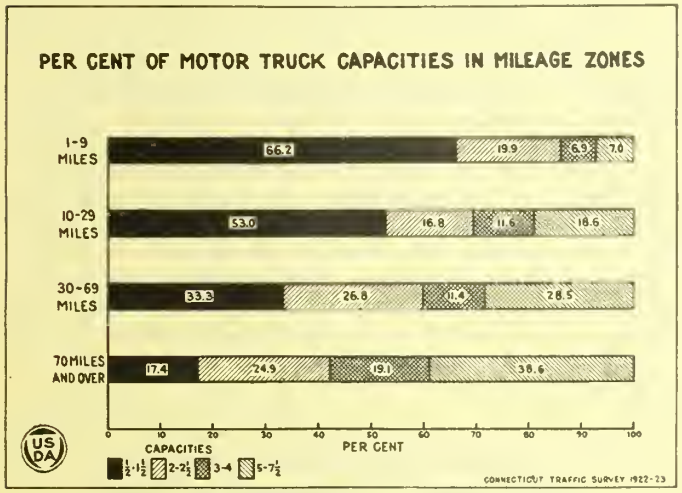


FIG. 25.—Truck capacities used in several mileage zones.

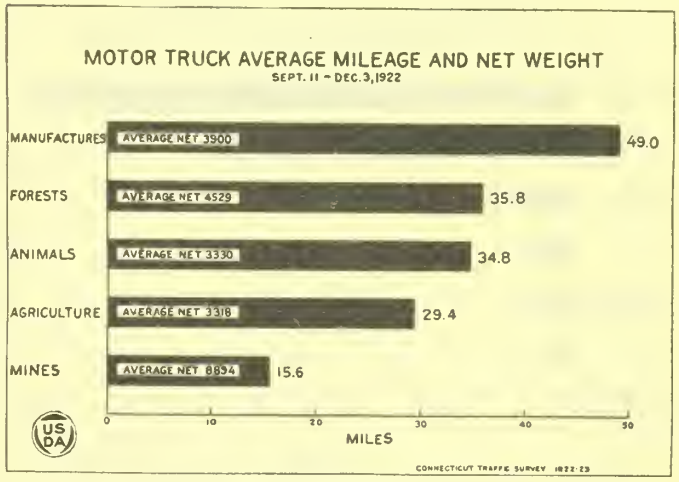


FIG. 28.—The average haul of manufactured commodities is greater than that of other commodities.

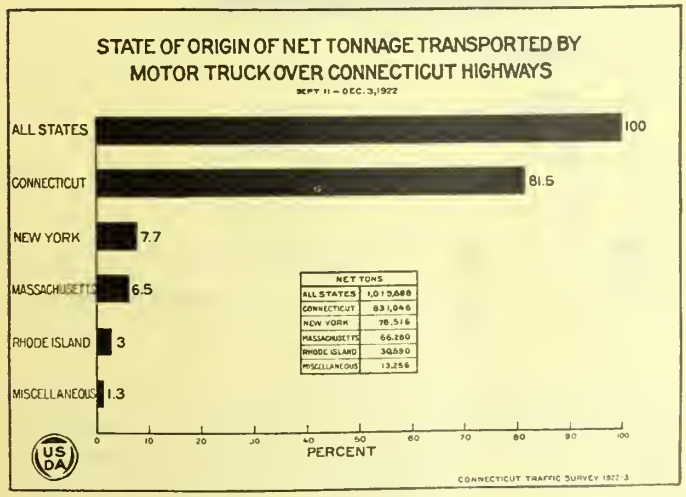


FIG. 26.—The bulk of commodities transported originates in Connecticut.

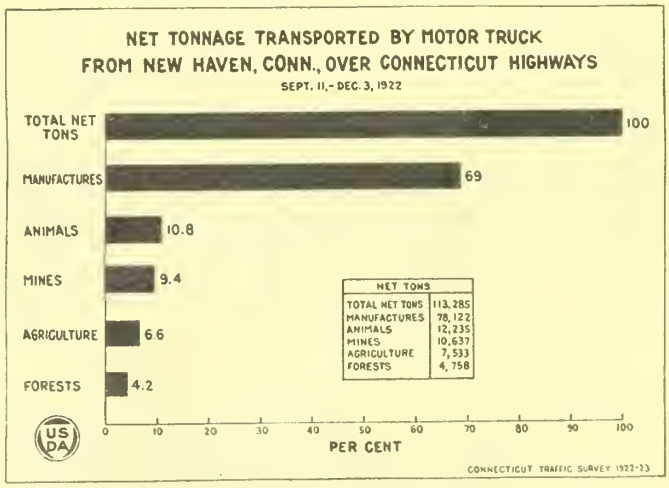


FIG. 29.—New Haven ranks first in origination of highway tonnage.

truck would be manufactured products. There is, however, a more pertinent explanation of the fact that 73.6 per cent of the commodities transported over the highways of this State are manufactured articles. Manufactured commodities are usually of small bulk and high value. The margin of profit in highway transportation of freight of this character is considerably larger than in the transportation of the heavy and bulky products of forests and mines.

Analysis of the origin and destination of motor-truck freight between cities and areas in New England indicates a tendency of outgoing and incoming net tonnage to balance. The small cities as a rule receive more truck freight than they originate, largely consumption goods. More motor-truck freight is shipped out of large centers of population than is received. This is largely due to a concentration of manufacturing near centers of population and the assembly of rail freight at larger centers for redistribution to smaller communities.

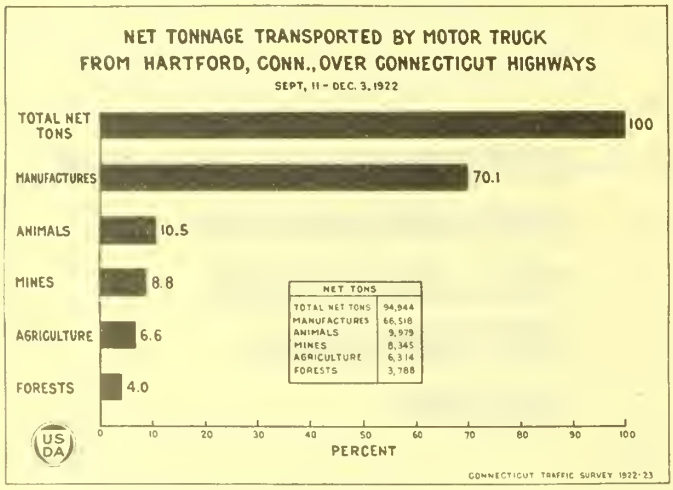


FIG. 30.—Hartford ranks second in origination of highway tonnage.

New Haven ranks first, Figure 29; Hartford second, Figure 30; and New York City third, Figure 31, in the origination of net tonnage transported by motor trucks over Connecticut highways. It is to be noted that the New Haven and Hartford percentage of tonnage for each commodity class is approximately the same, indicating that from cities of this size approximately the same ratio of commodity tonnage will be distributed by motor truck.

The influence of New York City as a motor-truck distribution center for products of agriculture and animals is indicated by the fact that while products of animals rank second and products of agriculture fourth for New Haven and Hartford, products of agriculture are second and products of animals third in importance for New York City.

THE MAJOR FIELDS OF MOTOR-TRUCK TRANSPORTATION.

A brief preliminary statement of the four main fields of operation for motor trucks in the transportation of freight is made in the order of their importance, based largely upon the results of the Connecticut Highway Transportation Survey supplemented by a study of motor-transport organization, operation, rates, and competition in the New England area.

1. Organized urban motor-truck transportation in congested rail and steamship terminal areas consisting of motor truck terminal-to-terminal freight transfers as well as pick-up-and-delivery service.

The congestion of incoming and outbound freight will be reduced, freight cars will be unloaded more rapidly and will be more efficiently utilized in the line haul of freight, and consignors and consignees of freight will receive better service. This is well illustrated by the operation of the United States Trucking Co. in New York City cooperating with the Erie Railroad in transferring freight by motor truck from the Erie, New Jersey, terminal to New York City for delivery. The success of organized terminal trucking depends upon the formation of terminal trucking companies with an organization large enough to handle efficiently the volume of business and guarantee satisfactory service.

Carefully organized and efficiently operated terminal trucking companies, assured of the cooperation of the rail and steamship operators, will materially speed up the terminal movement of freight, decrease the volume of freight warehoused, reduce the cost of moving freight through terminals, and, above all, lower the rail delivery time of L. C. L. freight. Such a cooperative arrangement will result in rapid terminal-to-terminal transfers and pick-up-to-delivery freight service, at least in the larger cities, guarantee a reasonable profit to the terminal motor-trucking company and expedite the movement of freight in congested areas. The organization of one large urban motor trucking company, or affiliation of a limited number of large trucking companies, will stabilize the motor-trucking business, permit the making of uniform rates, insure a fair return, eliminate the irregular "cut-throat" operator, and tend to reduce street congestion in terminal areas.

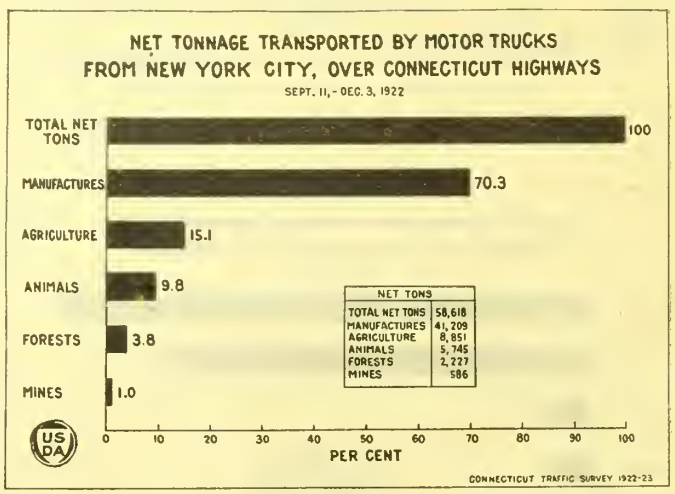


FIG. 31.—New York City ranks third in origination of highway tonnage in Connecticut.

2. The organization of motor-truck freight service supplementing existing rail and water transportation systems. The development of motor-trucking companies in areas inadequately served with rail or water transportation offers an enormous possibility for the economic extension of highway transportation. This service is especially desirable in the economic development of new areas or localities with insufficient trans-

portation facilities and will make available additional tonnage for movement by rail or water. This is a noncompetitive service, extending and supplementing existing rail and water transportation.

3. The short-haul transportation of freight in the New England area, approximately 30 miles. Transportation by motor truck in this zone is largely noncompetitive, consisting mainly of the assembly and distribution of goods.

The short-haul zone in one area is not necessarily applicable to another area. The distances between cities and areas of production and distribution, the type of prevailing production as well as the character of rail or water transportation may decrease or increase the above zone of the short haul.

4. Truck transportation of special commodities in the long-haul zone in which delivery time, the character of the goods transported, or demands of the industry or trade indicate the desirability of motor-truck transportation. This type of freight is but a small percentage of the total net tonnage transported by motor truck.

MOTOR-TRUCK MOVEMENT LARGELY NONCOMPETITIVE.

In general, the analysis of motor-truck transportation in the New England area indicates that the bulk of the shipment, considered on a tonnage basis, is a noncompetitive, short-haul transportation of freight. From the manufacturers' point of view, "with a permanent improvement in rail service a considerable portion of the long-haul motor-truck freight will be shipped by rail." Motor trucking at present is loosely organized, keenly competitive, operating largely on a contract basis, with rapid fluctuation in rates and with a gradual development of larger motor-trucking companies which will ultimately insure stability of service and rates. A difficult problem is met in applying the principle of government regulation of common carriers to motor-truck companies engaged in the transportation of freight in New England. At the present time approximately 75 per cent of the motor-truck tonnage is transported under contract agreements between the shippers and motor-truck operators. These contracts, oral or written, fix the rate per hundred pounds of freight for definite periods, and the rates may vary for each individual shipper. The regulation by governmental agencies of motor-truck transportation of freight under agreements of this nature may be open to doubt. This question, however, has not been finally passed upon by the courts and until it is so decided there can be no authoritative determination of it. Motor-truck operating companies are compelled to operate largely under the contract system on account of the destructive competition of competing motor-truck companies. It is entirely probable that with the development of governmental regulation of motor-truck transportation, with the resulting stabilization of the motor-truck rate structure and publication of rates, the majority of motor-trucking companies now operating in New England will discontinue the contract system. Motor-truck operators, according to their own statements, "would welcome reasonable regulation which will insure stability of rates and eliminate the 'cut-throat' competition" which now prevails in the New England area.

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One of the outstanding developments of the present era of motor-truck transportation in New England is the sharp competition existing between interurban motor-truck operators. They do not operate on the basis of published tariffs since rates fluctuate rapidly, largely due to the underbidding of small motor-trucking organizations and irregular operators. There is a lack of uniformity as to the basis of motor-truck rate making. Rail rates, mileage, tonnage available, "what the traffic will bear," contracts, bid business, and the probability of return loads are the principal factors in rate making. The return-load business is an important factor in rate making, and as a general practice in New England the rates charged for return loads are lower than rates for outbound freight on the theory that a small profit on the return trip is more profitable than to return empty. The result of this practice is to lower the level of rates charged for outbound business. For example, motor-truck companies operating out of New York City to New Haven, Conn., are forced to meet the competition on their outbound New York-New Haven trip of the lower rates of the New Haven motor trucks returning from New York City. The New Haven operators on outbound loads to New York City must meet the return-trip rates of the New York City trucks returning from New Haven. The result is that the practice of lower rates for return-trip cargoes forces down the rates on outbound loads and the trend is for rates to be influenced largely by the lowest return-load rates charged which tend to approach the cost of operation.

MANUFACTURERS PREFER TO HIRE TRUCKS.

The relatively high percentage of failure, or withdrawal from business, of the smaller trucking companies indicates a narrow margin of profit. Business depression or a decrease in the volume of freight available for truck transportation results in intense competition for business, lowered rates, and a transfer of trucking companies to other fields.

As a general rule manufacturers find it unprofitable to own trucks for L. C. L. transportation, preferring to hire local trucking companies, usually on a contract basis. A combined motor truck and boat, and motor truck and rail L. C. L. transportation service is developing in the New England territory. The Starin-New Haven Boat Line transports freight from Hartford, Meriden, Middletown, New Britain, Wallingford, and Waterbury, Conn., to New Haven by motor truck and by boat from New Haven to New York City. Freight is shipped on the basis of joint or through rates. The same type of service on a smaller scale is offered by the Hanson Steamship Co. from Norwalk, Conn., to New York City, and the North & East River Steamship Co. from Stamford, Conn. to New York City.

The development of combined motor-truck and water service is a practicable possibility in the northeast territory in view of the fact that the majority of the industrial cities are situated within a maximum distance of 40 miles from available water-shipping points. Combined truck and water transportation between Connecticut points and New York City provides an overnight movement of freight as well as a pick-up-and-delivery service outside of port towns.

COMBINED MOTOR-TRUCK AND RAIL SERVICE DEVELOPING.

A correlation of motor-truck and rail service is developing similar to joint motor-truck and water transportation of freight. The service performed by Stone's Motor Truck Express is typical of this development. Freight is assembled by motor truck in eastern New England territory tributary to Providence, Pawtucket, and Woonsocket, R. I., and Boston, Lynn, Salem, Peabody, Cambridge, Worcester, Fitchburg, Leomister, Brockton, and Haverhill, Mass. Freight in this area is transported by motor truck to selected rail shipping points and the long haul is handled by the New York, New Haven & Hartford Railroad. On an average, 10 carloads of freight are shipped daily from this territory by Stone's Express to New York City. Stone's Express ship freight according to the above methods in territory tributary to Philadelphia, Pa., Baltimore, Md., Washington, D. C., Camden, N. J., and the Eastern Shore of Maryland, in cooperation with the Pennsylvania Railroad.

The Boston & Springfield Despatch Trucking Co. offer a combined motor-truck and rail service between Boston and Springfield, Mass., in cooperation with the Boston & Albany Railroad. The New York & New England Despatch Motor Truck Co. operate combined motor-truck and rail freight service between New York City and Springfield, Mass., in cooperation with the New York Central and Boston & Albany Railroads.

This modern correlation of highway transportation with rail and water agencies for the movement of freight is an economic function of motor transportation and offers an extensive field for development, supplementing and extending the facilities of rail and water service. It is not competitive, provides a pick-up-and-delivery service, allocates the short haul to motor transportation and the long haul to rail or water, and provides rapid transportation of L. C. L. freight from consignor to consignee, which is the aim of effective transportation.

REASONS GIVEN FOR MOTOR-TRUCK SHIPMENT.

Generally, manufacturers in New England ship by motor truck for two principal reasons:

(1) *Reliable and prompt service.*—Motor-truck delivery to a given destination is rapid and the consignment is certain to arrive on time. This is particularly true of consignments from inland points to docks for boat transportation.

(2) *Trade demands.*—The retail trade, appreciating the economy of maintaining a smaller stock of goods with a smaller outlay of invested capital and fresh goods, insist upon motor-truck delivery daily or often enough to replenish their supply.

The fact that motor-truck rates may be below the level of rail rates constitutes a factor secondary in importance to rapid delivery and trade demands in determining the manufacturers' method of shipping.

The additional cost of rail transportation, due to packing and crating charges and weight of packing and crating, is not an important factor in the manufacturers' choice of motor-truck transportation. As a general rule only a small proportion of the net tonnage transported in New England by motor truck is packed or crated materially different for rail or truck movement.

The volume of products shipped by New England manufacturers by motor truck is small compared with their rail L. C. L. and C. L. tonnage. The attitude of the majority of manufacturers can be summed up as follows: "If service becomes efficient and rapid the most of our freight will be shipped by rail." A lower truck rate per 100 pounds is not the determining factor. It is largely a question of service time. Certain types of commodities are especially adapted to motor-truck transportation. In the majority of cases the character of the commodity, the volume regularly available for shipment, and the service offered determine the method of transportation.

Railroad freight congestion and rail embargoes force manufacturers to ship goods by motor truck in both the long and short haul. Rail embargoes result in an immediate increase in the net tonnage of freight transported over the Connecticut highway system.

The improvement of rail service in New England, particularly beyond the 30-mile haul, has resulted in a decrease in the use of motor-truck transportation in the long haul, except for a few specialized commodities in which trade considerations or the nature of the commodities make it desirable to use motor-truck transportation. Actual or potential competition of motor-trucking companies with rail or water service in the transportation of freight is an incentive to both rail and water operating companies to provide effective transportation.

Highway transportation is an integral part of our transportation system and the facts obtained from the Connecticut Highway Transportation Survey clearly illustrate that in making a comprehensive study of the transportation problems of the country it is essential to obtain a sufficient volume of highway traffic data in different areas to evaluate scientifically the economic services that highway systems produce in cooperation with rail and water agencies in the transportation of people and commodities.

THE FLOW OF WATER THROUGH PIPE CULVERTS.

COOPERATIVE TESTS BY THE U. S. BUREAU OF PUBLIC ROADS AND THE STATE UNIVERSITY OF IOWA.

A COMPARISON OF CONCRETE, VITRIFIED CLAY, AND CORRUGATED METAL PIPES.

By D. L. Yarnell, Senior Drainage Engineer, U. S. Bureau of Public Roads; and Sherman S. Woodward, Professor, and Floyd A. Nagler, Associate Professor, Department of Mechanics and Hydraulics, State University of Iowa.

A SERIES of tests, designed to supply greatly needed data on the flow of water through short pipes such as culverts, particularly of the larger diameters, has been made during the past year by engineers of the Bureau of Public Roads and the State University of Iowa under a cooperative arrangement entered into by the two agencies.[1]

The results, it is believed, will be of considerable general interest and may embody useful suggestions for the highway engineer.

Three facts stand out from the results of the tests as worthy of the most serious consideration of highway engineers.

The first is that highway engineers must pay more attention to the coefficient of roughness of the material forming the culvert. So long as the different materials used for culvert pipe did not differ greatly in roughness and hence in their frictional resistance to moving water, engineers were perhaps justified in not giving this factor much consideration. But in recent years a new material, corrugated metal, has been extensively manufactured into culvert pipe. Pipes made of this material are shown by these tests to offer much greater frictional resistance to the flow of water than other materials used, such as vitrified clay, cast iron, concrete, and timber. While for pipes of each material the coefficient of roughness in the Kutter formula is found to increase with increase in the size of pipe, the tests show that, for all sizes it is nearly twice as great for corrugated metal pipe as for concrete and vitrified clay pipe.

The second fact brought out clearly by the tests is that the quantity of water a culvert will discharge is directly proportional to the square root of the head and bears no relation to the grade at which the pipe is laid, if the pipe flows full, as it should at maximum capacity. The water in a pipe culvert under these conditions does not act as does that flowing in an open ditch where the quantity of discharge is dependent upon the slope or grade of the water surface in the ditch, but, as is the case with any pipe flowing full, the discharge depends upon the water pressure available to force the water through the opening and the pipe. In the case of a culvert the water pressure which causes discharge is that furnished by the difference between the water level at entrance and outlet. The depth of

FIG. 1.—Hydraulic laboratory looking downstream toward laboratory building. Testing canal with 24-inch corrugated metal pipe in place. Platform on left supports stilling well and hook gauge for obtaining head on the weir.

submergence has no effect on the discharge so long as the difference of the water levels at the two ends of the culvert remains the same.

The third outstanding observation is that the head loss at the culvert entrance is an important factor in determining the discharge and varies greatly with the type of entrance used. The data on the effect of different types of entrance on the entrance loss are among the most interesting of the findings from the tests.

For a square-cornered entrance the average entrance loss coefficient is found to be 0.393. Textbooks on hydraulics give the coefficient for a square-cornered entrance as 0.50.

METHODS OF REDUCING ENTRANCE LOSS.

The beveled lip of concrete pipe, shown in Figure 5, assists greatly in reducing the entrance loss at a straight end wall entrance, the average entrance loss coefficient for all sizes and lengths being 0.099.

A vitrified clay pipe culvert with the same form of straight end wall is especially efficient by virtue of the shape of the bell end of the pipe, shown in Figure 6. The average entrance loss coefficient for the clay pipe bell is 0.063. By taking care to make a gradually rounded entrance, this entrance loss can always be reduced practically to zero. When the bell is filled with concrete shaped in the form of an ellipse, the coefficient becomes 0.020.

Although the entrance loss is greater for a corrugated metal pipe, the reinforced rounded end of the metal pipe greatly assists in reducing the amount of entrance loss. The average entrance loss coefficient for the corrugated metal pipe is 0.226.

[1] The experiments were conducted by D. L. Yarnell, senior drainage engineer, U. S. Bureau of Public Roads, and Floyd A. Nagler, associate professor of mechanics and hydraulics, State University of Iowa, under the direction of S. H. McCrory, chief of the division of agricultural engineering, U. S. Bureau of Public Roads. Sherman M. Woodward, professor of mechanics and hydraulics, State University of Iowa, acted as consulting engineer for the investigations, making suggestions in the conduct of the experiments and collaborating in the preparation of the data and the report.

The tests on the vitrified clay and corrugated metal pipe, comprising 1,139 separate experiments, were run from Aug. 22 to Nov. 4, 1922. The tests on the concrete pipe, 341 in all, were run from Apr. 20 to May 29, 1923.

A. F. Fisher, highway engineer of Johnson County, Iowa, furnished the corrugated metal pipe used. The concrete and vitrified clay pipe were purchased by the U. S. Bureau of Public Roads.

Special conical entrances tested did not prove as efficient as the bell end of a clay pipe in reducing the entrance loss, although they assist greatly in reducing the entrance loss into a corrugated metal pipe. The average entrance loss coefficient for the various conical entrances used in connection with vitrified clay pipe is 0.088 whereas for the clay pipe bell this coefficient is 0.063. The average entrance loss coefficient for the various conical entrances used in connection with corrugated metal pipe is 0.044, whereas for the regular end of a metal pipe this coefficient is 0.226. There is little difference in the effectiveness of the various types of conical entrances in reducing the entrance loss.

The standard commercial vitrified clay pipe increasers when used as entrances are not as effective in reducing the entrance loss as the regular bell end of a clay pipe. The average entrance loss coefficient for the standard commercial vitrified clay pipe increasers is 0.142, whereas for the regular bell of a vitrified clay pipe this coefficient is 0.063.

FIG. 2.—24-inch concrete pipe in place for testing.

The tests of a 45° wing wall used in connection with a corrugated metal pipe culvert show that it has a slightly beneficial effect in reducing the entrance loss below that resulting from the use of the straight end wall. The average entrance loss coefficient for these wings used with corrugated metal pipe culverts is 0.221. The efficiency of these wings in increasing the discharge in a metal pipe over that obtained with the straight end wall will be seen by comparing the discharge equations in Table 3.

A vitrified clay pipe culvert used with a 45° wing wall actually shows a greater entrance loss and a lesser discharge than the same pipe with the straight end wall. The average entrance loss coefficient for these wings used in connection with the clay pipe is 0.114, whereas for the straight end wall it is 0.063.

U-type wings are relatively inefficient compared with a straight end wall in reducing the entrance loss in vitrified clay pipe. The wings increase the entrance loss and consequently decrease the discharge of the culvert. The average entrance loss coefficient for the U-type wings is 0.197 compared with 0.063 obtained for clay pipe with the straight end wall.

Apparently for the concrete pipe it makes very little difference in the entrance loss whether the pipe projects 3 inches, 2 feet, or 4 feet beyond the head wall. When the 12-inch concrete pipe with a square corner projected 3 inches beyond the head wall, the entrance loss coefficient was 0.354; with a 2-foot projection the entrance loss coefficient was 0.342; and with a 4-foot projection, the entrance loss coefficient was 0.361. However, for a 4-foot projection of 12-inch concrete pipe with beveled lip entrance, the entrance loss coefficient was only 0.092.

For 18-inch corrugated metal pipe with a 3-inch projection the entrance loss coefficient was 0.314, while for the same pipe with 2-foot and 4-foot projections the entrance loss coefficients were about 0.55.

The carrying capacity of vitrified clay pipe culvert with a straight end wall may be increased somewhat by filling the bell with cement mortar, rounding the mortar so as to form an elliptical entrance.

The discharge of any pipe culvert having a square-cornered entrance may be increased by setting the pipe back a few inches from the face of the head wall and rounding the concrete in the head wall next to the circumference of the pipe.

While it is recognized that practical considerations will dictate how far the hydraulic principles involved in these results can be followed, the application of the first set of results, namely, those showing the coefficient of roughness of the several types of pipe used for highway culverts, is both direct and obvious.

DISCHARGE A FUNCTION OF SQUARE ROOT OF HEAD FOR FULL CULVERT.

The conclusion that the discharge of a culvert is proportional to the square root of the head and bears no relation to the grade at which the pipe is laid is of course based upon the assumption that the culvert is designed to flow full, an assumption which probably departs from the prevalent practice, but it is believed that there are sound reasons for altering present methods of design in this particular.

If highway culverts are so designed and installed that they are only partially filled at times of great run-off, one naturally asks just how deep should a culvert be filled to discharge the water for which it is designed? This depth might form the basis for a "capacity" rating for comparing different types of culverts. On the other hand, if a culvert under such conditions discharges only partially full, is there not great lack of economy in the design which gives a larger waterway than necessary? After considerations such as these the writers concluded that the normal basis for comparing culverts should be established upon the pipe flowing full, giving its maximum capacity with a given "head." It is certain that most pipe culverts, even 18 or 24 inches in diameter, although installed to drain only a small area, may at some time be taxed to their full capacity. For instance, a run-off of 1 inch in 20 minutes from 2 acres of land will entirely submerge an 18-inch pipe at the inlet. It seems only logical, therefore, that the full culvert should be the capacity basis for design. However, the comparisons which have been made in this report apply without modification to culverts flowing half full or any fraction of full depth.

It is recognized that an obstacle to the design of culverts as pipes flowing full is presented in the inclination of highway engineers to keep the culvert high and near the road surface in order to reduce the quantity of ditching at the outlet end. The writers believe that sometimes this procedure reduces the discharging

capacity of the culvert below what might easily be obtained by merely lowering the culvert at slight expense under favorable conditions. They can see no danger in installing a culvert so that it will flow full if possible. The "head" or pressure involved is not as dangerous as it might seem. Flowing under a "head" is intended to mean merely flowing full, not flowing under any excessive head.

The statement that the discharge bears no relation to the grade at which the pipe is laid under the conditions assumed implies no objection on the part of the writers to the laying of culverts on slight grades. Culverts are laid on a slope in order that they may clear themselves of sediment during normal flow. This is a matter that has not been under investigation in these tests and hence is not emphasized. But it is the desire of the authors to emphasize that it is not *necessary* to lay a culvert on a slope to develop maximum capacity, i. e., flowing full; in fact, it is quite doubtful if the practice of laying the culvert on a grade has all the merits claimed for it. Certainly in flat country the culvert grade does not determine the velocity of flow but rather the difference in elevation of the water at the upstream and downstream ends. Even when flowing partially full, the water surface will find the grade it needs for flow through the culvert irrespective of the slope of the bottom of the culvert.

If the culvert is installed on a steep grade the chances that it will flow full and hence discharge at maximum capacity are greatly reduced. The writers have performed experiments which demonstrate that the slope is not very great at which the discharge capacity of a culvert, even with the inlet end submerged, will be considerably less for a given difference in water surface upstream and downstream than for the same culvert built at a flatter grade.

The question has been raised as to whether the significance of the nature of the entrance would be as great in the case of pipes flowing less than full as in the experiments with pipes flowing full under head. It has been suggested that under the conditions normally obtaining in pipe culvert installations the width of the approach ditch is greater than the diameter of the pipe and that these conditions might result in the formation of eddies which would vitiate the effect of the rounded pipe entrance. It is possible to settle whatever doubt there may be on this score. Experiments have been made with culverts flowing partially full and it has been demonstrated that the nature of the entrance is fully as significant as when the pipe flows full, if not more so. The experimental channel was itself several times wider than the culverts and experimental results when eddies were formed could not be distinguished from those obtained when the water above the culvert seemed perfectly quiet.

PREVIOUS EXPERIMENTS AND METHODS OF DESIGN.

Nearly 30 different formulas have been proposed for use in determining the run-off and waterways required for culverts. In practically all of these formulas the area of the waterway is given direct. Apparently no consideration has been given to the coefficient of roughness in the culvert pipe. Although it might appear that, since the amount of run-off as determined is only an approximation, there is no necessity for considering the roughness factor in the culvert; yet the great variation in this factor for culverts of various materials would seem to warrant some consideration of this coefficient. Most formulas contain a variable whose value depends upon the topography of the watershed tributary to the proposed culvert.

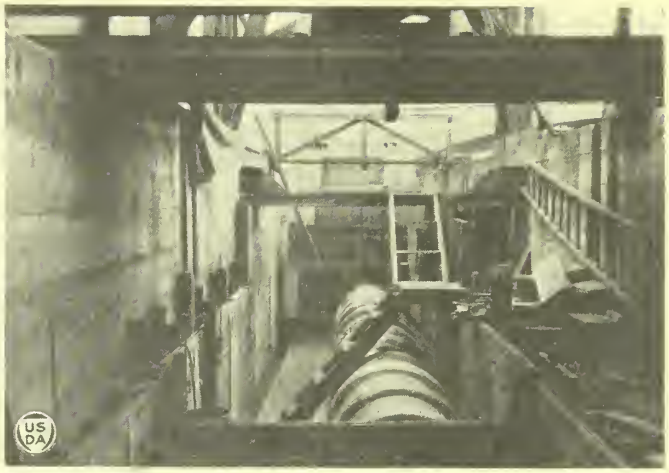

FIG. 3.—30-inch vitrified clay pipe in place for testing.

Of the various highway departments and railroads, apparently only the Pennsylvania Railroad engineers [2] use the well-known Kutter formula for determining the required area of waterway. They compute the volume of water reaching the site of the proposed culvert in a given time by the Burkli-Ziegler formula.

This formula is

$$q = cr\sqrt[4]{\frac{s}{a}}$$

in which q = the water reaching the culvert in cubic feet per second per acre.
r = the average cubic feet of rainfall per second per acre during heaviest rainfall.
s = the general grade of the drainage area in feet per thousand.
a = the area drained in acres.
c = a coefficient. For average areas, c equals 0.625.

Gilman and Chamberlain [3] state that the Burkli-Ziegler formula was brought to this country by Rudolph Hering in 1881.

The formula most commonly used for computing the velocity of flow in open channels and pipes is

$$V = C\sqrt{Rs}$$

in which V = velocity in feet per second.
R = mean hydraulic radius, or cross-sectional area of flow divided by wetted perimeter, in feet.
s = the grade or slope in feet per foot of length.
C = a coefficient.

[2] "Determining Sizes of Culverts," O. L. Grover, Public Roads, Vol. 1, No. 12, April, 1918, p. 39.
[3] "The Principal Formulas Proposed for Determining Run-Off and Waterways for Culverts," Engineering Contracting, Vol. XXV, Mar. 29, 1911, p. 366.

This formula was first proposed by a French engineer, Chezy, in 1775. Universal experience shows that the coefficient C varies with the hydraulic radius, the slope, and the amount of frictional resistance offered by the walls of the pipe. The measure of this frictional resistance is commonly called the coefficient of roughness or the roughness factor. Ganguillet and Kutter, two Swiss engineers, in 1869 derived an expression for computing the coefficient C in Chezy's formula based upon experimental data in open channels. Their formula takes into consideration the effect of the slope, the coefficient of roughness, and the hydraulic radius. This formula is so complicated that it is seldom used directly in hydraulic computations. Instead, diagrams and tables based on the formula are used almost exclusively by practicing engineers to obtain numerical results corresponding to the formula. Although numerous other formulas have been proposed, probably the great majority of engineers in English-speaking countries still use Kutter's formula.

Engineers heretofore have not considered the coefficient of roughness in pipes or conduits of short lengths to be of much importance. This, as has been stated already, was perhaps natural and justifiable so long as the different materials used for culvert pipe did not differ greatly in roughness and hence in their frictional resistance to moving water. In recent years, however, corrugated metal has come into extensive use for culvert pipe, forming a conduit whose surface obviously causes a greater frictional resistance to the flow of water than the other materials used, such as vitrified clay, cast iron, concrete, and timber.

Many tests have been made to determine the coefficient of roughness of concrete and vitrified clay pipes as well as pipes of other materials, but comparatively few have been conducted on corrugated metal pipe. Probably the first tests [4] on the flow of water in corrugated metal pipe were made by Cone, Trimble, and Jones in 1913. These tests were made on a semicircular metal flume having an arc length of 132 inches and a lineal length of 1,745 feet. The coefficient of roughness "N" for use in Kutter's formula varied from 0.0196 to 0.027, depending upon whether the tests were made on a tangent or a curve.

In 1917, the division of drainage investigations, Bureau of Public Roads, U. S. Department of Agriculture, conducted a series of experiments on the flow of water in two sizes of corrugated metal pipe, 8 and 10 inches in diameter. The length of pipe tested was 200 feet. The coefficient of roughness "N" obtained for the pipe flowing full ranged from 0.017 to 0.021. A synopsis of these tests was published in [5] Engineering News-Record.

SCOPE OF THE INVESTIGATION.

In order to determine accurately the effect of friction on the flow of water in short pipes, a total of 1,480 tests were made on concrete, vitrified clay, and corrugated metal pipe culverts flowing partly full and full, both with a free and submerged outlet. The sizes tested of each kind were 12, 18, 24, and 30 inches in diameter. To determine the effect of the length of the culvert on the flow, the 24-inch pipe of the three kinds of material was tested in three lengths, namely, 24, 30, and 36 feet. The other sizes were tested in the 30-foot length only.

It has been demonstrated that in pipes of short length the loss of head at the entrance of the pipe is an important factor. Whereas many tests have been made on small orifices acting under both low and high heads, few tests have been made on large circular vertical orifices acting under low heads. In culverts this latter condition prevails. For the purpose of studying the effect of various types of entrance in reducing the entrance loss several types of entrance were used. These included all of the standard types approved by the U. S. Bureau of Public Roads which consist of the straight end headwall, wing walls set at 45° to the pipe line, and the U-type wing. The effect of varying the height of the wing wall was also tested. In addition to these entrances a study was made of the flow through pipe culverts constructed without any head walls. This was accomplished by projecting the entrance end of the pipe for some distance through the headwall.

The maximum range of head obtained on the culverts for the different sizes tested varied from 1.05 feet on the 30-inch clay pipe to 3.29 feet on the 12-inch clay pipe.

DESCRIPTION OF EXPERIMENTAL PLANT.

The hydraulic laboratory of the State University of Iowa [6] is located on the west bank of the Iowa River, south of the university dam. The laboratory, built in 1919, consists of three main parts—the testing canal, the basin, and the tailrace. For this investigation only the testing canal was used. This canal (fig. 1), built of concrete, is 130 feet long, 10 feet wide, and 10 feet deep. At the upstream end and connected with the dam is a wooden gate 10 feet wide by 12 feet deep. This gate is regulated by a hand-operated hoist. Just below the gate, in the wall next to the river, a wooden sluice gate, 5 feet wide by 4 feet high, is provided for the purpose of flushing out silt which may collect above the dam near the intake. Recesses were built in the walls of the canal at intervals of 25 feet for use in building wooden bulkheads when needed in experimental work. Every 10 feet along the canal and 1 foot above its bottom, 2-inch pipes were placed transversely through the east wall for the attachment of piezometer tubes. At the lower end of the testing canal another 5 by 4 foot wooden sluice gate is set in the east wall. The floor of the testing canal has a grade of 0.20 per cent sloping to the south.

THE WEIRS.

For use in measuring the amount of water flowing through the pipes, a sharp-crested rectangular weir of the suppressed type was constructed. The length of the weir crest used in the experiments on the 24 and 30 inch pipe was 10 feet. Since, for the smaller pipe, it was necessary to obtain smaller quantities of water than could be measured on the 10-foot weir using the lowest head consistent with accuracy, a false wall was constructed upstream from the weir bulkhead for a distance of 36 feet, thus making possible the use of a weir with a crest of 5 feet.

The weir plate was so placed that the nappe of the weir cut free and was fully aerated by means of the recesses in the walls of the flume at each end of the crest.

[4] "Frictional Resistance in Artificial Waterways," by V. M. Cone, R. E. Trimble, and P. S. Jones, Colorado Agricultural Experimental Station Bulletin No. 194, 1914, p. 9.

[5] "The Coefficient of Roughness in Corrugated Iron Pipe," D. L. Yarnell, Engineering News-Record, vol. 88, Mar. 2, 1922, p. 352.

[6] "State University of Iowa's New Hydraulic Laboratory," by Stuart Sims, Engineering News-Record, vol. 85, July 15, 1920, p. 124.

The quantity of water passing over the weir was regulated by raising or lowering the headgate at the upper end of the canal. Several baffles were built in the canal in order to insure that the water, entering the canal with much turbulence, would pass over the weir in a smooth condition and with a fairly uniform velocity distribution.

To determine the discharge over the weir, Bazin's formula was used in all computations. This formula is:

$$Q=\left[0.405+\frac{0.00984}{H}\right]\left[1+0.55\left(\frac{H}{p+H}\right)^2\right]b\sqrt{2g}\,H^{\frac{3}{2}}$$

in which Q = discharge in cubic feet per second.
H = head in feet on the weir.
b = length in feet of crest.
p = height in feet of weir crest above the bottom of the flume.
g = acceleration of gravity = 32.16 feet per second.

The quantity in the first bracket represents a coefficient and the quantity in the second bracket is the correction for the velocity of approach to the weir.

A bear-trap weir, 4 feet high and located 18 feet downstream from the outlet of the 36-foot culvert, was used to obtain a submerged outlet for the pipe. This weir was hung on hinges and was regulated by means of a block and tackle attached to a windlass which was mounted on a platform built over the canal.

HOOK GAUGES.

Five hook gauges were used in conducting the investigation. These were of the all-metal Gurley type with a 45° point.

Hook gauge No. 1 was located on the east side of the canal 15.77 feet from the weir. The gauge was bolted to a heavy block which was securely fastened to the outer side of the concrete wall of the canal. A 1½-inch pipe connected the opening in the wall of the canal, 10¾ inches above the bottom, to a 15 by 36 inch cylindrical galvanized stilling tank placed immediately under the hook gauge. This opening corresponded quite closely to that used by Bazin in his noted weir experiments; Bazin's opening was 16.35 feet from the weir and 6 inches above the floor of the canal.

Hook gauge No. 2 was located near the upper end of the pipe line being tested. This gauge was used to obtain the elevation of the surface of the water in the canal at the entrance to the pipe. The gauge was bolted to a block which was secured to the west wall of the canal. A 1½-inch pipe led from the opening in the bulkhead of the culvert being tested to another galvanized stilling well placed directly under the gauge.

Hook gauge No. 3 was placed near the bulkhead to the outlet of the 24-foot length of pipe. It was mounted in a manner similar to gauge No. 2. This gauge was installed to measure the water level in the canal at the outlet.

Hook gauge No. 4 was located at the outlet of the 30-foot pipe.

Hook gauge No. 5 was located at the outlet of the 36-foot pipe.

PIEZOMETERS AND PIEZOMETER TUBES.

In order to measure the depth of flow in the culvert when flowing partly full, as well as to secure the hydraulic gradient when the outlet of the pipe was submerged, glass piezometer tubes fastened to enameled staff gauges were placed on the side of the canal and connected to the underside of the culvert pipe as described below.

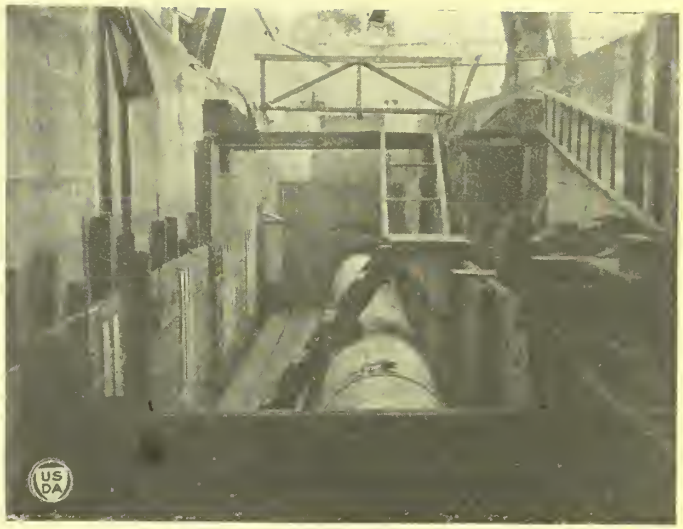

FIG. 4.—30-inch corrugated metal pipe in place for testing.

At the mid-point of each joint of vitrified clay pipe a small hole was drilled through its wall and a ¼-inch iron nipple, 3½ inches long, was inserted, care being taken that the tube did not project inside the tile bore. This tube was set in cement and any unevenness on the inside wall of the pipe at the entrance of the tube was removed by coating the surface with a little cement. The piezometer connections in the concrete pipe were made in a similar manner.

In making the piezometer connections in the corrugated metal pipe spacings were used which conformed as nearly as possible to those of the concrete and clay pipe. Holes were drilled in the surface of the outer corrugation on the outside of the pipe and ¼-inch iron nipples, 3½ inches long, were inserted through these holes until the nipples were flush with the inside bore of the pipe. The space in the corrugation around the nipple inside the pipe was filled with solder, thus forming a plane surface of some extent on all sides of the orifice. This type of piezometer connection was considered best for obtaining the pressure of the water in corrugated metal pipe after experiments on several types of piezometer connections in this kind of pipe.

The culvert pipe were placed so as to have the tubes about 8 inches above the inside bottom of the pipe. Connections were made by rubber tubing to 1-inch glass tubes 3 feet long attached to white enameled gauge staffs secured to the side of the canal. These gauge staffs, 3.3 feet long, were graduated with divisions of 0.02 foot, and the markings were such that they could be easily read with little chance of error. With glass tubes of 1-inch bore, the effect of capillarity is negligible and need not be considered.

LAYING THE PIPE.

The pipes were laid level with water-tight joints on a wooden floor in the bottom of the testing canal. (See figs. 2, 3, and 4.) Tight wooden bulkheads were installed at each end of the pipe line, so that the pipe could remain exposed in the dry while running full of water even with both ends submerged to a considerable depth. The entrance to the culvert pipe was located 26.8 feet downstream from the weir. Two baffles were

placed in the canal upstream from the pipe in order to avoid commotion of the water as it entered the pipe.

The joints of the concrete pipe were made water-tight with cement mortar.

The joints of the vitrified clay pipe were made water-tight as follows: From two to three strands of oakum were placed in the bell around the circumference of the pipe and tightly tamped. The remaining space was then filled with cement mortar, which was packed in the bell. It was necessary to allow the mortar to set overnight to obtain sufficient strength to resist the pressure of the water.

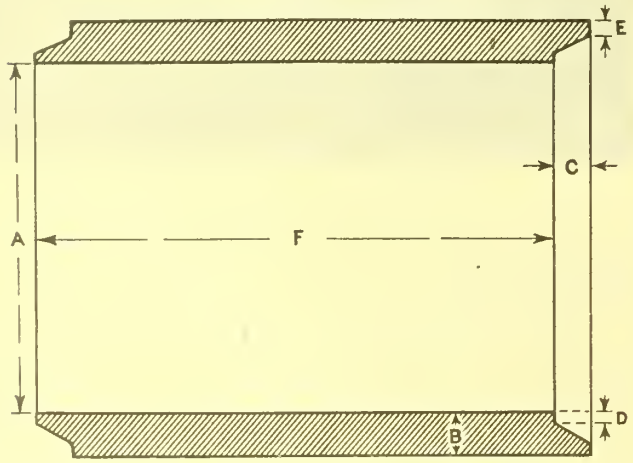

FIG. 5.—Longitudinal section of concrete pipe with beveled-lip joint.

The joints between two successive lengths of corrugated metal pipe were made water-tight by the following method: Strands of oakum were wrapped around the ends of each section in two different corrugations. The collar was then attached and drawn up tightly by means of bolts. The joints were then filled with melted pitch, which, when solidified, gave a water-tight joint. The seams of the individual pipe were soldered.

In testing the various conical entrances each cone was inserted in the inlet end of the culvert pipe, and a false head wall of 1-inch boards was constructed flush with its upstream end. The various wing walls tested were connected directly to the main head wall at the inlet end of the culvert.

DESCRIPTION OF PIPE USED.

Although in actual practice the nominal or commercial size of culvert pipe is always used in computing the cross-sectional area, to determine accurately the retardation factors it was essential in these tests to know the correct average diameter of all pipe tested. For this purpose two diameters at right angles to each other were measured at each end of every pipe used.

These measurements showed that the concrete and the corrugated metal pipe measured practically the nominal or commercial size, while the vitrified clay pipe generally were a little larger than the nominal size. A longitudinal section of the concrete pipe is shown in Figure 5, and a longitudinal section of the vitrified clay pipe tested is given in Figure 6. The dimensions of the concrete pipe are given in Table 1, and those of the clay pipe in Table 2.

TABLE 1.—*Dimensions of concrete pipe.*[1]

A	B	C	D	E	F
Inches.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*
12	0.167	0.13	0.06	0.05	2.00
18	.23	.21	.06	.06	2.00
24	.25	.21	.06	.08	3.00
30	.29	.25	.09	.10	3.00

[1] Letter designations refer to Figure 5.

TABLE 2.—*Dimensions of vitrified clay pipe.*[1]

A	B	C	D
Inches.	*Feet.*	*Feet.*	*Feet.*
12	0.09	0.20	0.12
18	.11	.24	.15
24	.13	.28	.18
30	.20	.34	.29

[1] Letter designations refer to Figure 6.

HYDRAULIC ELEMENTS INVOLVED IN DETERMINING CAPACITY OF PIPE CULVERTS.

In order to determine the coefficient of roughness in the culvert pipe and the formula for the carrying capacity of a culvert of any length, the magnitude of each hydraulic element involved in the actual discharge of the culvert must be obtained by experiment. The elements to be determined are: (1) The mean velocity of the water in the culvert; (2) the hydraulic gradient in the pipe; (3) the head lost at the culvert entrance.

Mean velocity.—The method of determining the mean velocity of the water flowing in the pipes involved the measurement of the quantity of water entering the culvert per second, and then solving the relation, *mean velocity equals quantity divided by area of cross section.*

Hydraulic gradient.—The hydraulic gradient was obtained by means of piezometers. Tests were made on the pipe both with a free and with a submerged outlet. When water flows through any culvert, the hydraulic gradient will take the slope required to overcome the retarding effect of the friction acting along the walls of the pipe.

Head lost at the culvert entrance.—The head lost at the culvert entrance is an important factor in the discharge of a culvert and varies greatly with the type of entrance used. This loss in head is determined as fol-

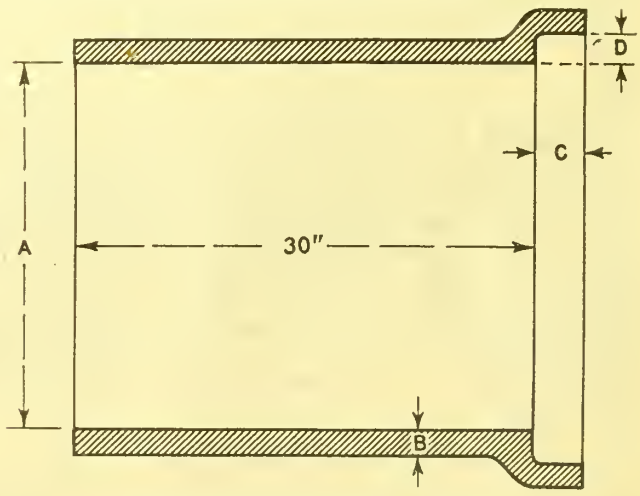

FIG. 6.—Longitudinal section of vitrified clay pipe.

lows: The difference of the elevation of the water surface at the culvert inlet and the elevation of the hydraulic gradient at this point as determined by the piezometer readings is computed. From this amount, called the entrance drop, the velocity head is subtracted, and the remainder is the loss in head for the type of entrance used.

METHOD OF CONDUCTING TESTS.

Tests on each culvert were begun with a head of 0.30 foot of water discharging over the measuring weir, followed by experiments with successive increases of 0.10 foot in head on the weir, until the maximum quantity was obtained which would flow through the pipe without overtopping the bulkhead at the entrance to the culvert. For each head on the measuring weir several different tests were secured by manipulating the bear-trap weir, as follows: (1) With the outlet discharging freely into the air; (2) with the elevation of the water surface at the culvert outlet raised to the middle of the pipe; (3) with the water surface at the outlet brought up to 6 inches below the top of the pipe; (4) with the water surface raised to the top of the pipe at the outlet; (5) with the water surface at the culvert outlet raised by successive increases of 6 inches until the maximum possible submergence was secured.

In obtaining a series of readings for different stages with the outlet submerged and with a constant head on the weir, a set of runs was secured with a constant discharge and having hydraulic gradients approximately parallel in which certain hydraulic elements should check.

Since, in testing the various entrances, the aim was to obtain data on the efficiency of the different types used, only submerged runs were secured. In this case, the first test was begun with a head of 0.4 foot on the weir and was followed by successive increases of 0.1 foot until the maximum quantity was obtained. For each head three different depths of submergence were taken.

An electric bell was placed on the derrick directly over the culvert being tested and was connected with another bell placed on the observation platform at the weir hook gauge. The push button for operating these bells was located at hook gauge No. 2 at the entrance to the culvert pipe.

To secure the data necessary for determining the hydraulic elements for each kind of pipe tested, five men were required, to act in the following capacities:

(1) Observer of piezometers.
(2) Observer at hook gauge No. 1 located at the weir.
(3) Observer at hook gauge No. 2 located at the culvert entrance.
(4) Observer at hook gauge No. 3 located at the culvert outlet.
(5) Man for operating head-gate and bear-trap weir.

PROGRESS OF THE TEST.

From start to finish, a single test progressed as follows:

(1) The head gate was adjusted to obtain the required head on the weir.

(2) The stage of the water surface at the entrance of the culvert was watched to determine when it became steady.

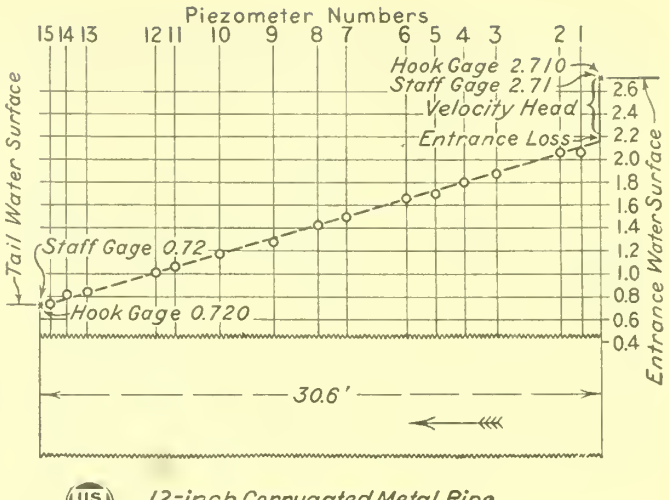

FIG. 7.—Profile of 12-inch corrugated metal pipe with straight end wall entrance.

(3) The test was then ready to begin. Tests were numbered consecutively. The observer at hook gauge No. 2 gave the time of starting to the observer of hook gauge No. 3 and to the observer of piezometers, both of whom recorded this time above the number of the test on their log charts. Two short bells announced that a test was to begin.

(4) Ten seconds later, one short bell gave the time to take the first reading.

(5) One bell every 30 seconds signaled simultaneous readings on all the hook gauges, during which period the piezometers were read.

(6) Readings were continued until 10 consecutive readings on each hook gauge were secured.

(7) Three bells denoted the close of a test.

(8) The observer at hook gauge No. 3 directed the operator to raise the bear-trap weir for the next test.

In having the observer of the piezometers secure single readings on all hook gauges, a check was obtained on the reading of these gauges. Error in reading a gauge is much more apt to occur in the reading of the 0.1 foot than in either the 0.01 or the 0.001 foot. No check was necessary on the readings of the piezometers, as the observer could not possibly retain all of the readings for the first set (15 to 30 in number, depending upon the length of the pipe) in his mind when taking the second set.

As soon as possible after the conclusion of each test the average values of all the different readings were plotted on a diagram which was, in effect, a condensed profile of the pipe under test. (See fig. 7.) Near the right margin of the diagram is shown the headwater level above the entrance to the pipe; at the left margin similarly is shown the tail water level at the outlet of the culvert. Along the pipe in their proper respective positions are shown the elevation of the water in the various piezometer tubes. The diagram shows at once the amount of submergence of each end of the culvert pipe and the total differences in water level between the bays at the two ends of the pipe.

Through the points representing the water level in the various piezometer tubes an average straight line was drawn, as shown in Figure 7. This line represents the hydraulic gradient in the pipe. The diagram shows strikingly that, while at the outlet end of the pipe, if submerged, the hydraulic gradient nearly meets the tail

water level—frequently it is slightly below it—on the other hand, at the entrance end of the pipe the hydraulic gradient is often far below the headwater level. The slope of the hydraulic gradient indicates the true friction loss within the pipe.

DISCUSSION OF EXPERIMENTAL RESULTS.

The results of 1,192 tests contained in 58 separate tables (not included in this article) have been studied in detail and from these studies certain conclusions have been drawn.

Effect of depth of submergence.—A study was first made to determine, for a culvert submerged at both ends, whether any definite relation exists between the depth of submergence and the ratios of entrance loss to velocity head and friction loss to velocity head. It was found that these ratios are practically constant, and thus independent of the amount of submergence of the culvert. At the same time it was found that unless the entrance to the pipe was submerged to some extent the pipe would not flow full, and thus would not achieve its full discharging capacity.

Relation of losses to velocity.—To determine the relation between the mean velocity in the pipe and the ratios of entrance loss, friction loss, and total head to velocity head, the mean velocity, mean entrance drop, mean friction loss, and mean total head were determined for each group of tests in which the head on the weir was approximately the same for all the tests in different tables. The average velocity head was computed from the mean velocity for each group. The entrance loss was obtained by subtracting from the mean entrance drop the new velocity head, and this loss divided by the new velocity head to determine the

TABLE 3.—*Summary of results of test data.*

Table No.	Pipe. Kind.	Pipe. Size.	Pipe. Length.	Remarks.	Entrance loss coefficient.	Ratio of velocity head to total head.	Ratio of friction loss to total head.	Ratio of entrance loss to total head.	Equation.
		Inches	*Feet.*			*Per cent.*	*Per cent.*	*Per cent.*	
12	Concrete	12	30	Straight end wall entrance—Pipe with beveled lip	0.111	52.8	41.6	5.6	$Q = 4.62\ H^{0.470}$
13	...do	18	30	.....do	.097	64.8	29.8	5.4	$Q = 11.42\ H^{0.484}$
14	...do	24	30	.....do	.093	69.9	25.9	4.2	$Q = 20.8\ H^{0.470}$
15	...do	30	30	.....do	.137	74.1	20.7	5.2	$Q = 33.1\ H^{0.474}$
16	...do	12	30	Straight end wall entrance—Pipe with square end	.276	49.0	37.6	13.4	$Q = 4.42\ H^{0.486}$
17	...do	18	30	.....do	.404	53.6	25.4	21.0	$Q = 10.3\ H^{0.482}$
18	...do	24	30	.....do	.437	55.7	20.3	24.0	$Q = 18.65\ H^{0.478}$
19	...do	30	30	.....do	.493	61.2	14.7	24.1	$Q = 29.8\ H^{0.467}$
20	Clay	12	30	Straight end wall entrance	.010	64.8	34.7	0.5	$Q = 5.25\ H^{0.482}$
21	...do	18	30	.....do	.044	68.5	29.5	2.0	$Q = 11.9\ H^{0.505}$
22	...do	24	30	.....do	.095	69.6	25.9	4.5	$Q = 22.25\ H^{0.481}$
23	...do	30	30	.....do	.120	72.6	21.0	6.4	$Q = 32.0\ H^{0.485}$
24	Metal	12	30	.....do	.160	24.1	72.1	3.8	$Q = 3.0\ H^{0.505}$
25	...do	18	30	.....do	.200	31.0	63.1	5.9	$Q = 7.86\ H^{0.498}$
26	...do	24	30	.....do	.225	39.6	52.2	8.2	$Q = 15.8\ H^{0.523}$
27	...do	30	30	.....do	.295	42.6	45.8	11.6	$Q = 24.9\ H^{0.485}$
28	Concrete	24	24	Straight end wall entrance—Pipe with beveled lip	.080	72.0	23.8	4.2	$Q = 21.0\ H^{0.488}$
29	...do	24	36	.....do	.061	68.7	30.4	.9	$Q = 20.45\ H^{0.485}$
30	Clay	24	25	Straight end wall entrance	.122	70.1	22.0	7.9	$Q = 22.3\ H^{0.496}$
31	...do	24	38	.....do	.057	68.4	29.8	1.8	$Q = 21.8\ H^{0.489}$
32	Metal	24	24	.....do	.298	42.9	44.4	12.7	$Q = 16.5\ H^{0.505}$
33	...do	24	36	.....do	.230	36.2	55.8	8.0	$Q = 15.1\ H^{0.505}$
34	...do	24	36	Straight end wall entrance (ascending series)	.212	36.0	56.7	7.3	$Q = 15.05\ H^{0.481}$
35	...do	24	36	Straight end wall entrance (descending series)	.236	34.8	57.3	7.9	$Q = 14.8\ H^{0.500}$
36	...do	24	36	Straight end wall entrance with floor in front of entrance	.436	34.0	50.1	15.9	$Q = 14.75\ H^{0.512}$
37	Clay	24	38	Conical entrance, 13° angle, 10 inches long	.110	67.8	26.6	5.6	$Q = 21.7\ H^{0.477}$
38	...do	24	38	Conical entrance, 13° angle, 20 inches long	.027	69.4	30.7	−.1	$Q = 21.7\ H^{0.473}$
39	...do	24	38	Conical entrance, 45° angle, 10 inches long	.105	67.2	27.0	5.8	$Q = 21.8\ H^{0.481}$
40	Metal	24	36	Conical entrance, 13° angle, 10 inches long	.043	38.5	60.1	1.4	$Q = 15.65\ H^{0.507}$
41	...do	24	36	Conical entrance, 13° angle, 20 inches long	.040	38.5	60.0	1.5	$Q = 15.65\ H^{0.508}$
42	...do	24	36	Conical entrance, 24° 47′ angle, 10 inches long	.050	38.6	60.2	1.2	$Q = 15.60\ H^{0.504}$
43	Clay	12	30	12-15-inch standard commercial increaser used as entrance	.128	61.2	31.6	7.2	$Q = 5.1\ H^{0.496}$
44	...do	12	30	12-18-inch standard commercial increaser used as entrance	.083	61.2	33.7	5.1	$Q = 5.1\ H^{0.496}$
45	...do	18	30	18-20-inch standard commercial increaser used as entrance	.190	62.3	27.2	10.5	$Q = 11.3\ H^{0.490}$
46	Metal	24	36	45° wings, full height, set flush with inside edge of pipe, without floor in front of entrance.	.168	37.7	56.4	5.9	$Q = 15.5\ H^{0.491}$
47	...do	24	36	45° wings, standard height, set flush with inside edge of pipe, without floor	.243	37.1	54.1	8.8	$Q = 15.3\ H^{0.492}$
48	...do	24	36	45° wings, full height, set 6 inches from inside edge of pipe, without floor	.224	36.6	55.5	7.9	$Q = 15.25\ H^{0.488}$
49	...do	24	36	45° wings, standard height, set 6 inches from inside edge of pipe, without floor.	.269	37.0	54.2	8.8	$Q = 15.25\ H^{0.482}$
50	Clay	24	38	45° wings, full height, set flush with inside edge of bell with floor in front of entrance.	.116	66.9	26.1	7.0	$Q = 21.75\ H^{0.475}$
51	...do	24	38	45° wings cut level to top of standard end wall and set flush with inside edge of bell, with floor in front.	.122	67.0	26.1	6.9	$Q = 21.55\ H^{0.476}$
52	...do	24	38	45° wings cut on bevel to top of standard end wall and set flush with inside edge of bell, with floor in front.	.111	66.9	26.0	7.1	$Q = 21.55\ H^{0.479}$
53	...do	24	38	45° wings, standard height, set flush with inside edge of bell, with floor in front of entrance.	.106	67.6	27.8	4.6	$Q = 21.60\ H^{0.479}$
54	Metal	24	36	45° wings, standard height, set flush with inside edge of pipe, with floor in front.	.365	35.7	51.7	12.6	$Q = 15.15\ H^{0.486}$
55	Clay	24	38	U-type wings cut on bevel to top of standard end wall and set flush with inside edge of bell, with floor in front of entrance.	.177	63.0	27.8	9.2	$Q = 21.05\ H^{0.483}$
56	...do	24	38	U-type wings cut on bevel to top of standard end wall and set 6 inches from inside edge of bell, with floor in front.	.150	65.1	25.6	9.3	$Q = 21.35\ H^{0.485}$
57	...do	24	38	U-type wings, standard height, set flush with inside edge of bell, with floor in front of entrance.	.291	58.6	26.9	14.5	$Q = 20.35\ H^{0.485}$
58	...do	24	38	U-type wings, standard height, set 6 inches from inside edge of bell, with floor in front of entrance.	.169	61.7	29.4	8.9	$Q = 20.70\ H^{0.476}$
59	...do	24	38	Straight end wall with entrance bell filled with concrete and surfaced off straight from inside edge of bell to inside edge of pipe.	.044	68.8	29.1	2.1	$Q = 22.05\ H^{0.490}$
60	...do	24	38	Straight end wall with entrance bell filled with concrete elliptically shaped with convex surface out.	.020	69.9	29.1	1.0	$Q = 22.2\ H^{0.485}$
61	...do	24	38	Straight end wall with entrance bell filled with concrete shaped to give a square-cornered entrance.	.478	54.8	20.7	24.5	$Q = 19.6\ H^{0.485}$
62	Concrete	12	31	3-inch projection beyond head wall—Pipe with square-cornered entrance	.354	45.8	38.0	16.2	$Q = 4.28\ H^{0.477}$
63	...do	12	31	24-inch projection beyond head wall—Pipe with square-cornered entrance	.342	46.5	37.8	15.7	$Q = 4.32\ H^{0.479}$
64	...do	12	33	47-inch projection beyond head wall—Pipe with square-cornered entrance	.361	45.4	38.4	16.2	$Q = 4.26\ H^{0.483}$
65	...do	12	33	47-inch projection beyond head wall—Pipe with beveled lip	.092	50.3	45.1	4.6	$Q = 4.47\ H^{0.482}$
66	Metal	18	36	3-inch projection beyond head wall	.314	27.1	64.6	8.3	$Q = 7.35\ H^{0.482}$
67	...do	18	36	24-inch projection beyond head wall	.552	25.8	60.1	14.1	$Q = 7\ 18\ H^{0.489}$
68	...do	18	36	48-inch projection beyond head wall	.568	25.3	60.5	14.2	$Q = 7.12\ H^{0.485}$
69	Clay	18	30	Straight end wall entrance 18 to 26 inch cone, 60 inches long at outlet end of pipe, length including cone 30 feet.	.032	64.3	33.3	2.4	$Q = 16.45\ H^{0.500}$

average entrance loss ratio. The mean friction loss ratio and mean total head ratio were obtained by dividing the mean friction loss and the mean total head by the new velocity head.

These ratios were plotted on coordinate paper with mean velocity in feet per second as abscissæ and the ratios of entrance loss, friction loss, and total head to velocity head as ordinates.

A study of these diagrams shows that the ratios of the entrance loss, friction loss, and total head to the velocity head are substantially constant, with a slight tendency toward increasing values for the higher velocities. For those tests in which the velocity fell below 3 feet per second the values of the ratios were generally markedly smaller than for the other tests. Since the data in such cases, however, probably were not as precise as in the other cases, it can not be definitely stated whether this apparent tendency indicates a real law or is only due to unavoidable errors in the measurements. The fact that these ratios were so nearly constant in each table proves that for any given installation the friction loss, the entrance loss, and the total loss are practically proportional to the square of the velocity of the water flowing through the pipe. In other words, the discharging capacity of a pipe is proportional to the square root of the head on the pipe.

Drop down at a free outlet.—When the culvert pipe was acting under some head and with a free outlet, it was noted that there was a drop down in the water surface at the outlet of the pipe. The amount of this drop in the water surface below the top of the pipe at the outlet varied both with the size of the pipe and the head on the pipe. This drop was measured in each case.

Methods of reducing entrance losses.—Among the many interesting things revealed by a study of the 58 tables of experimental data is the effect of different types of entrance on the entrance loss. The entrance loss coefficient has been averaged for each type of entrance and kind of pipe (see column 6, Table 3), excluding all tests in which the pipe velocity was less than 2.50 feet per second. The reason that velocities less than 2.50 feet per second were ignored in finding averages was that for such low velocities the values are less accurately determined than in the other cases. Negative entrance loss values were included and added algebraically. The average shown in Table 3 is a weighted average obtained by dividing the sum of the entrance losses by the sum of the velocity heads.

VALUES OF THE COEFFICIENT OF ROUGHNESS IN THE KUTTER AND MANNING FORMULAS.

The values of the coefficient of roughness in the Kutter and Manning formulas obtained for the individual tests have been averaged for the various pipes.

Since for heads of less than 0.40 foot on the weir the Bazin formula, used for determining the discharge over sharp-crested rectangular weirs of the suppressed type, gives quantities somewhat greater than those obtained by volumetric measurement, the values of the coefficient of roughness for tests with heads of less than 0.40 foot may be a little less than the correct ones. Therefore in obtaining the average values for the coefficient of roughness in the tables of the experimental results all tests with heads of less than 0.40 foot on the weir have been omitted. The average values of the coefficient of roughness in the Kutter and Manning formulas for the various sizes of the three kinds of pipe tested are shown in Table 4.

TABLE 4.—*Average values of the coefficient of roughness in concrete, vitrified clay, and corrugated metal culvert pipe.*

Diameter of pipe.	Kutter coefficient.			Manning coefficient.		
	Concrete.	Clay.	Metal.	Concrete.	Clay.	Metal.
Inches.						
12	0.0117	0.0101	0.0194	0.0119	0.0098	0.0228
18	.0121	.0119	.0217	.0121	.0118	.0248
24	.0130	.0127	.0216	.0130	.0125	.0239
30	.0127	.0131	.0232	.0125	.0131	.0254

This table shows that the coefficient of roughness in the Kutter formula is nearly twice as great for the corrugated metal pipe as for the concrete and vitrified clay pipe. The coefficient of roughness increases with an increase in the size of the pipe. The Manning coefficient for the metal pipe is also approximately twice the value for the concrete and clay pipe, and likewise increases with an increase in the size of the pipe.

The variation of the different values of the Manning coefficient is about the same as that of the Kutter coefficient, showing that throughout the range covered by these tests and so far as indicated by these results, the Manning coefficient is at least as satisfactory as the Kutter coefficient. For the smoother pipe the Kutter and Manning coefficients are practically the same as they are supposed to be. It must be noted, however, that for the rougher pipe the Manning coefficient is definitely higher than the Kutter coefficient, so that for this case it is not safe to use the coefficients as exactly interchangeable. Most coefficients increase for the larger diameters of pipes, indicating that neither the Kutter formula nor the Manning formula makes the correct allowance for change in diameter of pipe. The results indicate that in the Manning formula the value two-thirds, used as the exponent of the hydraulic radius, is too large.

Figures 8, 9 and 10 show the flow of the water in the different kinds of pipe.

COMPARISON OF CARRYING CAPACITY OF CONCRETE, VITRIFIED CLAY, AND CORRUGATED METAL PIPE CULVERTS.

The data presented in Table 3 constitute a condensed summary of the hydraulic elements most useful in making comparisons of the various culvert pipes.

The ratios of velocity head, friction loss, and entrance loss to the total head on the pipe have been determined and expressed in percentages. These percentages, as given in Table 3, are the averages for each kind of pipe for all heads of 0.40 foot and over on the measuring weir. Negative entrance losses have been included and added algebraically.

The ratios of velocity head, friction loss, and entrance loss to the total head on the pipe were plotted on rectangular cross-section paper as ordinates against the respective sizes of pipe as abscissæ. These curves are shown in Figure 11. They are of special importance, as they show that of the total head on the pipe, the percentage consumed in entrance loss and velocity head increases with an increase in the size of the pipe, whereas the percentage required by friction loss decreases with an increase in the size of the pipe. On account of this fact the relative difference in the carrying capacity of a 30-inch vitrified clay and a 30-inch corrugated metal pipe culvert under the same head is much less than the relative difference in the carrying capacity of a 12-inch vitrified clay and a 12-inch corrugated metal pipe culvert.

From the data in the tables of the experimental results, a discharge equation for each table was derived for the various kinds of culvert pipe. By plotting on logarithmic paper the total head on the pipe for each test as abscissa against discharge as ordinate, an expression, $Q = KH^x$, was obtained in which K is the intercept on the unity vertical axis and x is the slope of the line. These individual discharge equations are given in Table 3 opposite their respective table numbers.

It will be noted from an examination of these individual discharge equations that the exponent of H varies from 0.467 to 0.523, 11 being 0.500 or over and 47 being less than 0.500. The average of the exponents for all the concrete pipe with the beveled-lip end upstream is 0.478. The average exponent for the concrete pipe with a square-cornered entrance is also 0.478. The average exponent for the vitrified clay pipe with the bell-end upstream is 0.491. The average

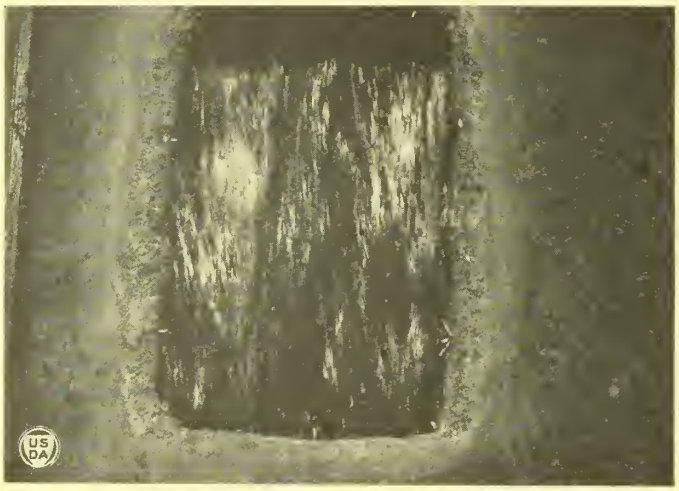

FIG. 8.—Water flowing in 24-inch concrete pipe. Depth of flow 1.62 feet, discharge 19.09 second-feet. Note smooth lines of flow.

exponent for the corrugated metal pipe is 0.504. These four averages are for the pipe culverts with straight end wall entrances only. The average of the exponents for the 58 tables of experimental results is 0.488. Since this average is nearly 0.500 and since it is common practice to accept the theory that the discharge varies as the square root of the head, it was decided to adopt the value of 0.500 as the exponent of H for subsequent calculations.

The exponent of H for the individual discharge equations varies somewhat. The value of the intercepts for the individual equations depends upon the slope of the line which is represented by the exponent of H. The values obtained for the intercepts when the slopes of the lines representing the individual pipe formulas were changed to 0.500 will vary somewhat and it was decided to use approximately the intercepts obtained in the original formulas in deriving the general discharge equations applicable to pipe culverts of any size.

DISCHARGE EQUATIONS DEVELOPED.

It was found that in laying the pipe culverts, the average length was approximately 30.6 feet, so this

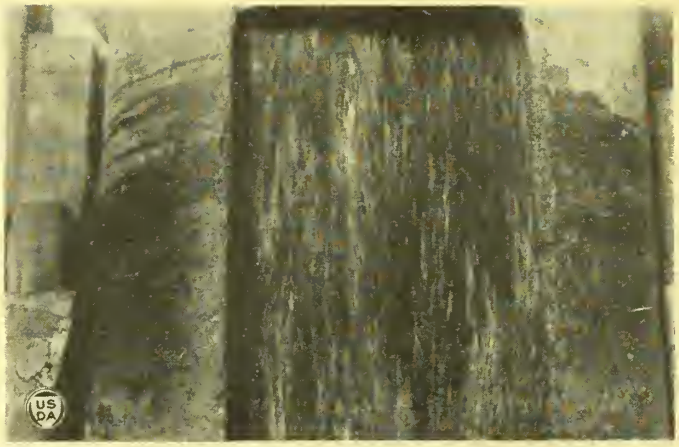

FIG. 9.—Water flowing in 24-inch vitrified clay pipe. Depth of flow 1.82 feet, discharge 20.95 second-feet.

figure was adopted as a base. Therefore, for the purpose of obtaining a comparison of the carrying capacities of various sizes of concrete, vitrified clay, and corrugated metal pipe culverts having the same length, discharge equations have been derived from the experimental data by the theory of least squares.

These general discharge equations for pipe culverts, 30.6 feet long with straight end-wall entrances are as follows:

Concrete pipe with beveled-lip end upstream.

$$Q = 4.61\ D^{2.18}\ H^{0.50} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Concrete pipe with square-cornered entrance.

$$Q = 4.40\ D^{2.09}\ H^{0.50} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Vitrified clay pipe with regular bell end upstream.

$$Q = 5.07\ D^{2.05}\ H^{0.50} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Corrugated metal pipe.

$$Q = 3.10\ D^{2.31}\ H^{0.50} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which Q = discharge in cubic feet per second.
D = diameter of pipe in feet.
H = total head on pipe or the difference in the water level at the two ends of the pipe.

By computing the friction loss per unit of length for each size and kind of pipe, and determining by the theory of least squares the equations involving this

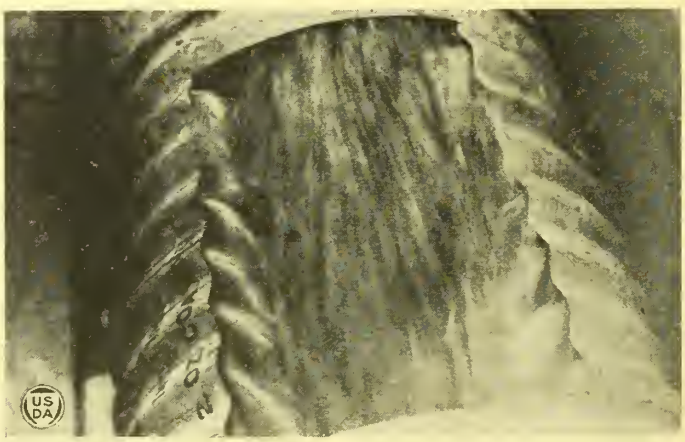

FIG. 10.—Water flowing in 24-inch corrugated metal pipe. Depth of flow 1.82 feet, discharge 17.88 second-feet. Note disturbance of filaments of flow due to corrugations.

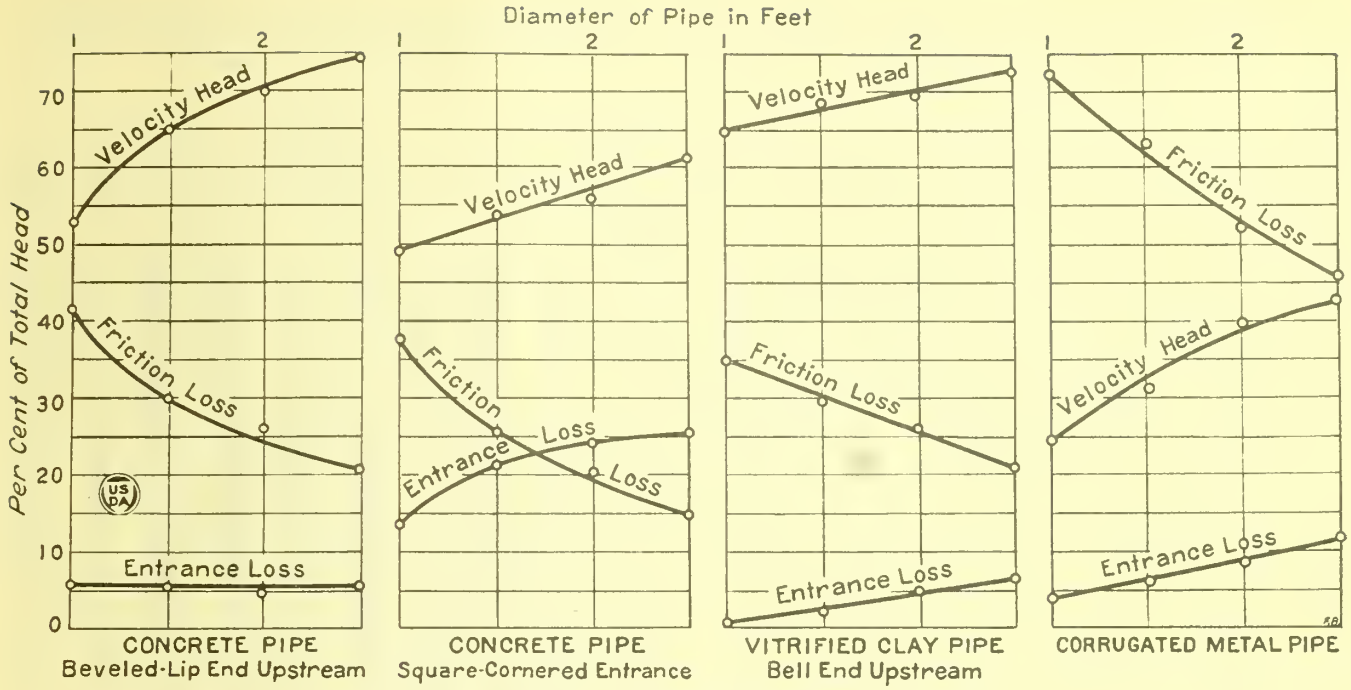

FIG. 11.—Losses in pipe culverts with straight end wall entrances. Length, 30 feet.

factor and the diameter of pipe, discharge formulas have been derived for concrete, vitrified clay, and corrugated metal pipe culverts of any length.

These formulas, which are applicable to lengths between 20 and 60 feet, are as follows:

Concrete pipe with beveled-lip end upstream:

$$Q = \frac{4.61\ D^{2.18}\ H^{0.50}}{\sqrt{1 + \frac{0.0138\ (L - 30.6)}{D^{0.756}}}} \quad \ldots\ldots \quad (5)$$

Concrete pipe with square-cornered entrance:

$$Q = \frac{4.40\ D^{2.09}\ H^{0.50}}{\sqrt{1 + \frac{0.0127\ (L - 30.6)}{D^{1.01}}}} \quad \ldots\ldots \quad (6)$$

Vitrified clay pipe with bell end upstream:

$$Q = \frac{5.07\ D^{2.05}\ H^{0.50}}{\sqrt{1 + \frac{0.0117\ (L - 30.6)}{D^{0.526}}}} \quad \ldots\ldots \quad (7)$$

Corrugated metal pipe:

$$Q = \frac{3.10\ D^{2.31}\ H^{0.50}}{\sqrt{1 + \frac{0.0240\ (L - 30.6)}{D^{0.489}}}} \quad \ldots\ldots \quad (8)$$

To represent the experimental values of discharge by one formula for each material which gives the proper weight and effect to the size and length of pipe is a very difficult matter. The usual formula for flow in pipes was tried. The type of this formula is as follows:

$$Q = \frac{\sqrt{2gH}\ A}{\left[1 + K_e + \frac{fL}{D^x}\right]^{\frac{1}{2}}} \quad \ldots\ldots \quad (9)$$

in which Q = discharge in cubic feet per second.
H = head on the pipe in feet.
A = area of pipe in square feet.
D = diameter of pipe in feet.
L = length of pipe in feet.
K_e = entrance loss coefficient.
f = friction loss coefficient.

It was at once apparent, however, that in these experimental results K_e is not a constant but varies with the diameter and length of the pipe. A partial trial of the results showed also that f and x are variable and can not be rigorously determined from these data. Further investigation will be necessary to determine whether a formula of the type of equation 9 can be used for flow in short pipes and what adaptations of it may be necessary. Because formula 9 is so complicated in form, a simple exponential type of formula was tried to see how it would represent the experimental results. Formulas 1 to 8 given above were found to represent the data throughout the range covered very satisfactorily.

Discharge tables have been computed from the general equations 1, 2, 3, and 4 for concrete, virtified clay, and corrugated metal pipe culverts for heads from 0.01 to 3.5 feet for the following sizes of pipe, 12, 15, 18, 21, 24, 30, 36, 42, and 48 inches in diameter. These capacities are shown in Tables 5, 6, 7, and 8.

For the purpose of enabling the highway engineer to determine the size of a pipe culvert of a certain kind when the quantity to be carried, the head on the pipe.

TABLE 5.—*Capacities of concrete pipe culverts, straight end wall entrance, length 30.6 feet, beveled-lip end upstream.*

[Discharge in cubic feet per second.]

Head on pipe.	Size of pipe.								
	12-inch.	15-inch.[1]	18-inch.	21-inch.[1]	24-inch.	30-inch.	36-inch.[1]	42-inch.[1]	48-inch.[1]
Feet.									
0.01	0.46	0.75	1.12	1.56	2.09	3.40	5.06	7.08	9.47
.02	.65	1.06	1.58	2.21	2.95	4.80	7.15	10.0	13.4
.03	.80	1.30	1.93	2.70	3.62	5.89	8.76	12.3	16.4
.04	.92	1.50	2.23	3.12	4.18	6.80	10.1	14.2	18.9
.05	1.03	1.68	2.49	3.49	4.67	7.60	11.3	15.8	21.2
.06	1.13	1.84	2.73	3.83	5.12	8.32	12.4	17.3	23.2
.07	1.22	1.98	2.95	4.13	5.53	8.99	13.4	18.7	25.1
.08	1.30	2.12	3.16	4.42	5.91	9.61	14.3	20.0	26.8
.09	1.38	2.25	3.35	4.68	6.27	10.2	15.2	21.2	28.4
.1	1.46	2.37	3.53	4.94	6.61	10.7	16.0	22.4	29.9
.2	2.06	3.35	4.99	6.98	9.34	15.2	22.6	31.6	42.3
.3	2.52	4.11	6.11	8.55	11.4	18.6	27.7	38.8	51.9
.4	2.92	4.74	7.06	9.88	13.2	21.5	32.0	44.8	59.9
.5	3.26	5.30	7.89	11.0	14.8	24.0	35.8	50.0	66.9
.6	3.57	5.81	8.64	12.1	16.2	26.3	39.2	54.8	73.3
.7	3.86	6.27	9.34	13.1	17.5	28.4	42.3	59.2	79.2
.8	4.12	6.71	9.98	14.0	18.7	30.4	45.2	63.3	84.7
.9	4.37	7.11	10.6	14.8	19.8	32.2	48.0	67.1	89.8
1.0	4.61	7.50	11.2	15.6	20.9	34.0	50.6	70.8	94.7
1.2	5.05	8.21	12.2	17.1	22.9	37.2	55.4	77.5	104
1.4	5.45	8.87	13.2	18.5	24.7	40.2	59.8	83.7	112
1.6	5.83	9.48	14.1	19.8	26.4	43.0	64.0	89.5	120
1.8	6.18	10.1	15.0	21.0	28.0	45.6	67.8	94.9	127
2.0	6.52	10.6	15.8	22.1	29.5	48.1	71.5	100	134
2.2	6.84	11.1	16.6	23.2	31.0	50.4	75.0	105	140
2.4	7.14	11.6	17.3	24.2	32.4	52.6	78.3	110	147
2.6	7.43	12.1	18.0	25.2	33.7	54.8	81.5	114	153
2.8	7.71	12.6	18.7	26.1	35.0	56.9	84.6	118	158
3.0	7.98	13.0	19.3	27.1	36.2	58.9	87.6	123	164
3.2	8.25	13.4	20.0	27.9	37.4	60.8	90.5	127	169
3.4	8.50	13.8	20.6	28.8	38.5	62.7	93.2	130	175
3.5	8.62	14.0	20.9	29.2	39.1	63.6	94.6	132	177

NOTE.—This table is based on the formula $Q=4.61\ D^{2.18}\ H^{0.50}$, in which Q=discharge in cubic feet per second, D=diameter of pipe in feet, and H=head on pipe in feet.

[1] No experiments were made on these sizes.

TABLE 7.—*Capacities of vitrified clay pipe culverts, straight end wall entrance, length 30.6 feet, regular bell end upstream.*

[Discharge in cubic feet per second.]

Head on pipe.	Size of pipe.								
	12-inch.	15-inch.[1]	18-inch.	21-inch.[1]	24-inch.	30-inch.	36-inch.[1]	42-inch.[1]	48-inch.[1]
Feet.									
0.01	0.51	0.80	1.16	1.60	2.10	3.32	4.82	6.61	8.69
.02	.72	1.13	1.65	2.26	2.97	4.69	6.82	9.35	12.3
.03	.88	1.39	2.02	2.77	3.64	5.75	8.35	11.5	15.1
.04	1.01	1.60	2.33	3.19	4.20	6.63	9.64	13.2	17.4
.05	1.13	1.79	2.60	3.57	4.69	7.42	10.8	14.8	19.4
.06	1.24	1.96	2.85	3.91	5.14	8.13	11.8	16.2	21.3
.07	1.34	2.12	3.08	4.22	5.56	8.78	12.8	17.5	23.0
.08	1.43	2.27	3.29	4.52	5.94	9.38	13.6	18.7	24.6
.09	1.52	2.40	3.49	4.79	6.30	9.95	14.5	19.8	26.1
.1	1.60	2.53	3.68	5.05	6.64	10.5	15.2	20.9	27.5
.2	2.27	3.58	5.21	7.14	9.39	14.8	21.6	29.6	38.9
.3	2.78	4.39	6.38	8.75	11.5	18.2	26.4	36.2	47.6
.4	3.21	5.07	7.36	10.1	13.3	21.0	30.5	41.8	55.0
.5	3.58	5.66	8.23	11.3	14.9	23.5	34.1	46.8	61.5
.6	3.93	6.21	9.02	12.4	16.3	25.7	37.3	51.2	67.4
.7	4.24	6.70	9.74	13.4	17.6	27.8	40.3	55.3	72.7
.8	4.53	7.16	10.4	14.3	18.8	29.7	43.1	59.1	77.8
.9	4.81	7.60	11.0	15.2	19.9	31.5	45.7	62.7	82.5
1.0	5.07	8.01	11.6	16.0	21.0	33.2	48.2	66.1	86.9
1.2	5.55	8.78	12.8	17.5	23.0	36.3	52.8	72.4	95.3
1.4	6.00	9.48	13.8	18.9	24.8	39.3	57.0	78.2	103
1.6	6.41	10.1	14.7	20.2	26.6	42.0	61.0	83.6	110
1.8	6.80	10.8	15.6	21.4	28.2	44.5	64.7	88.7	117
2.0	7.17	11.3	16.5	22.6	29.7	46.9	68.2	93.5	123
2.2	7.52	11.9	17.3	23.7	31.1	49.2	71.5	98.1	129
2.4	7.85	12.4	18.0	24.7	32.5	51.4	74.7	102	135
2.6	8.18	12.9	18.8	25.8	33.9	53.5	77.7	107	140
2.8	8.48	13.4	19.5	26.7	35.1	55.5	80.7	111	145
3.0	8.78	13.9	20.2	27.7	36.4	57.5	83.5	115	151
3.2	9.07	14.3	20.8	28.6	37.6	59.3	86.2	118	156
3.4	9.36	14.8	21.5	29.4	38.7	61.2	88.9	122	160
3.5	9.48	15.0	21.8	29.9	39.3	62.1	90.2	124	163

NOTE.—This table is based on the formula $Q=5.07\ D^{2.05}\ H^{0.50}$, in which Q=discharge in cubic feet per second, D=diameter of pipe in feet, and H=head on pipe in feet.

[1] No experiments were made on these sizes.

TABLE 6.—*Capacities of concrete pipe culverts, straight end wall entrance, length 30.6 feet, square-cornered entrance.*

[Discharge in cubic feet per second.]

Head on pipe.	Size of pipe.								
	12-inch.	15-inch.[1]	18-inch.	21-inch.[1]	24-inch.	30-inch.	36-inch.[1]	42-inch.[1]	48-inch.[1]
Feet.									
0.01	0.44	0.70	1.03	1.42	1.87	2.99	4.37	6.03	7.98
.02	.62	0.99	1.45	2.00	2.65	4.22	6.18	8.53	11.3
.03	.76	1.21	1.78	2.45	3.24	5.17	7.57	10.4	13.8
.04	.88	1.40	2.05	2.83	3.75	5.97	8.74	12.1	16.0
.05	.98	1.57	2.30	3.17	4.19	6.68	9.77	13.5	17.8
.06	1.08	1.72	2.52	3.47	4.59	7.32	10.7	14.8	19.5
.07	1.16	1.86	2.72	3.75	4.96	7.90	11.6	16.0	21.1
.08	1.24	1.98	2.90	4.01	5.30	8.45	12.4	17.1	22.6
.09	1.32	2.10	3.08	4.25	5.62	8.96	13.1	18.1	23.9
.1	1.39	2.22	3.25	4.48	5.92	9.44	13.8	19.1	25.2
.2	1.97	3.14	4.59	6.34	8.38	13.4	19.6	27.0	35.7
.3	2.41	3.84	5.62	7.76	10.3	16.4	23.9	33.0	43.7
.4	2.78	4.44	6.49	8.96	11.9	18.9	27.7	38.2	50.4
.5	3.11	4.96	7.26	10.0	13.3	21.1	30.9	42.7	56.4
.6	3.41	5.43	7.95	11.0	14.5	23.1	33.9	46.7	61.8
.7	3.68	5.87	8.59	11.9	15.7	25.0	36.6	50.5	66.7
.8	3.94	6.27	9.18	12.7	16.8	26.7	39.1	54.0	71.3
.9	4.17	6.65	9.74	13.4	17.8	28.3	41.5	57.2	75.7
1.0	4.40	7.01	10.3	14.2	18.7	29.9	43.7	60.3	79.8
1.2	4.82	7.68	11.3	15.5	20.5	32.7	48.0	66.1	87.4
1.4	5.21	8.30	12.2	16.8	22.2	35.3	51.7	71.4	94.4
1.6	5.57	8.87	13.0	17.9	23.7	37.8	55.3	76.3	101
1.8	5.90	9.41	13.8	19.0	25.1	40.1	58.6	80.9	107
2.0	6.22	9.92	14.5	20.0	26.5	42.2	61.8	85.3	113
2.2	6.53	10.4	15.2	21.0	27.8	44.3	64.8	89.5	118
2.4	6.82	10.9	15.9	21.9	29.0	46.3	67.7	93.5	124
2.6	7.10	11.3	16.6	22.9	30.2	48.2	70.5	97.3	129
2.8	7.36	11.7	17.2	23.7	31.4	50.0	73.2	101	133
3.0	7.62	12.2	17.7	24.5	32.5	51.7	75.7	105	138
3.2	7.87	12.6	18.4	25.3	33.5	53.4	78.2	108	143
3.4	8.11	12.9	18.9	26.1	34.5	55.1	80.6	111	147
3.5	8.23	13.1	19.2	26.5	35.1	55.9	81.8	113	149

NOTE.—This table is based on the formula $Q=4.40\ D^{2.09}\ H^{0.50}$, in which Q=discharge in cubic feet per second, D=diameter of pipe in feet, and H=head on pipe in feet.

[1] No experiments were made on these sizes

TABLE 8.—*Capacities of corrugated metal pipe culverts, straight end wall entrance, length 30.6 feet.*

[Discharge in cubic feet per second.]

Head on pipe.	Size of pipe.								
	12-inch.	15-inch.[1]	18-inch.	21-inch.[1]	24-inch.	30-inch.	36-inch.[1]	42-inch.[1]	48-inch.[1]
Feet.									
0.01	0.31	0.52	0.79	1.13	1.54	2.57	3.92	5.60	7.62
.02	.44	.73	1.12	1.60	2.17	3.64	5.55	7.92	10.8
.03	.54	.90	1.37	1.96	2.66	4.46	6.79	9.70	13.2
.04	.62	1.04	1.58	2.26	3.07	5.15	7.84	11.2	15.3
.05	.69	1.16	1.77	2.52	3.44	5.76	8.77	12.5	17.0
.06	.76	1.27	1.94	2.77	3.77	6.31	9.61	13.7	18.7
.07	.82	1.37	2.09	2.99	4.07	6.81	10.4	14.8	20.2
.08	.88	1.47	2.24	3.19	4.35	7.28	11.1	15.8	21.6
.09	.93	1.56	2.37	3.39	4.61	7.72	11.8	16.8	22.9
.1	.98	1.64	2.50	3.57	4.86	8.14	12.4	17.7	24.1
.2	1.39	2.32	3.54	5.05	6.87	11.5	17.5	25.0	34.1
.3	1.70	2.84	4.33	6.18	8.42	14.1	21.5	30.7	41.8
.4	1.96	3.28	5.00	7.14	9.72	16.3	24.8	35.4	48.2
.5	2.19	3.67	5.59	7.98	10.9	18.2	27.7	39.6	53.9
.6	2.40	4.02	6.13	8.75	11.9	19.9	30.4	43.4	59.1
.7	2.59	4.34	6.62	9.45	12.9	21.5	32.8	46.9	63.8
.8	2.77	4.64	7.07	10.1	13.8	23.0	35.1	50.1	68.2
.9	2.94	4.92	7.50	10.7	14.6	24.4	37.2	53.1	72.3
1.0	3.10	5.19	7.91	11.3	15.4	25.7	39.2	56.0	76.2
1.2	3.40	5.69	8.66	12.4	16.8	28.2	43.0	61.3	83.5
1.4	3.67	6.14	9.36	13.4	18.2	30.5	46.4	66.3	90.2
1.6	3.92	6.57	10.0	14.3	19.4	32.6	49.6	70.8	96.4
1.8	4.16	6.96	10.6	15.2	20.6	34.5	52.6	75.1	102
2.0	4.38	7.34	11.2	16.0	21.7	36.4	55.5	79.2	108
2.2	4.60	7.70	11.7	16.8	22.8	38.2	58.2	83.0	113
2.4	4.80	8.04	12.3	17.5	23.8	39.9	60.8	86.8	118
2.6	5.00	8.37	12.8	18.2	24.8	41.5	63.2	90.3	123
2.8	5.19	8.69	13.2	18.9	25.7	43.1	65.6	93.7	128
3.0	5.37	8.99	13.7	19.6	26.6	44.6	67.9	97.0	132
3.2	5.55	9.29	14.2	20.2	27.5	46.1	70.2	100	136
3.4	5.72	9.57	14.6	20.8	28.4	47.5	72.3	103	141
3.5	5.80	9.71	14.8	21.1	28.8	48.2	73.4	105	143

NOTE.—This table is based on the formula $Q=3.10\ D^{2.31}\ H^{0.50}$, in which Q=discharge in cubic feet per second, D=diameter of pipe in feet, and H=head on pipe in feet.

[1] No experiments were made on these sizes.

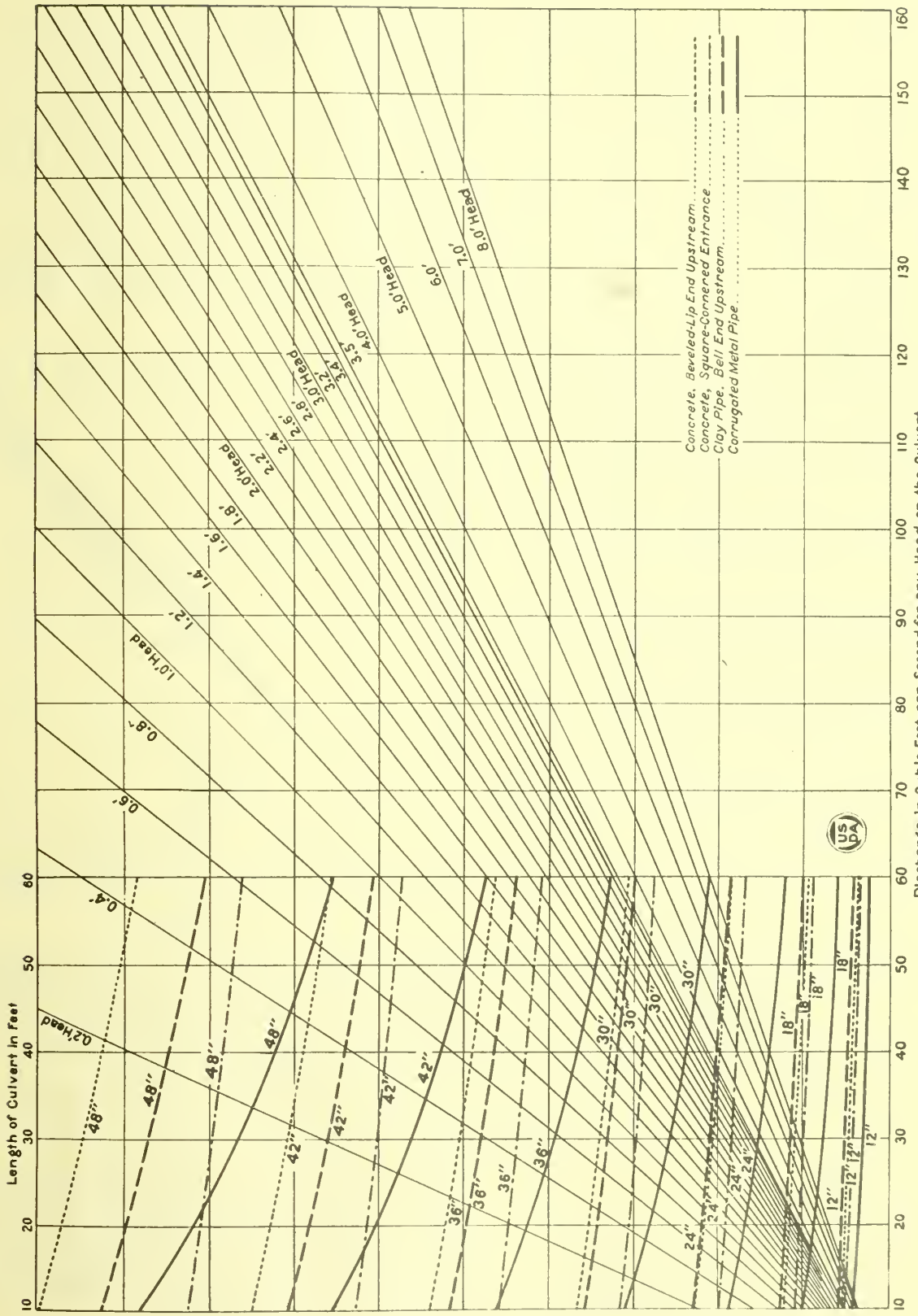

FIG. 12.—Diagram for determining the size of pipe culverts of any length required to carry a known quantity of water under a given head. For culverts with straight end wall entrances only.

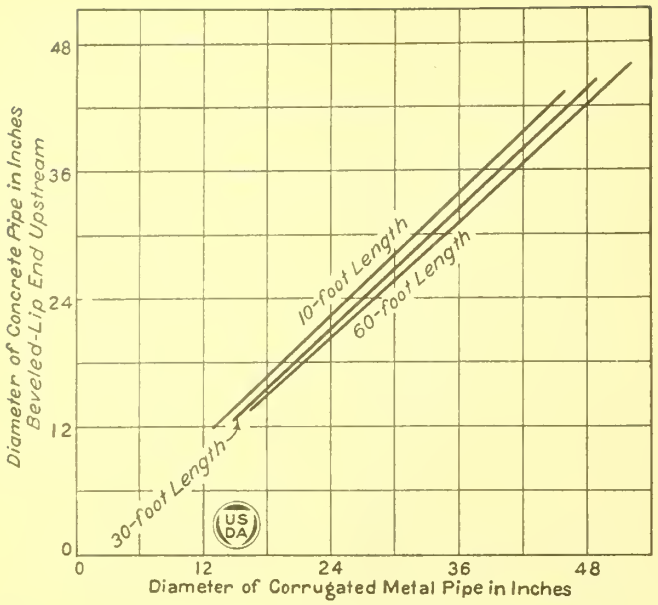

FIG. 13.—Diagram for concrete pipe culverts with beveled-lip end upstream and corrugated metal pipe culverts showing equivalent sizes when discharging the same quantity. Pipe acting under a head of 1 foot—straight end wall entrance.

and the length of the culvert are known, Figure 12 has been compiled from the discharge formulas deduced from the test data. This diagram is useful since no computations are required to obtain the desired information.

Discharge capacity in cubic feet per second for any head on the culvert has been plotted as abscissæ against discharge in cubic feet per second for a head of 1 foot on the culvert as ordinates. From the lower left-hand origin diagonal lines representing the different heads on the culvert have been drawn. On the left side of the diagram, lines representing different sizes of culverts have been established, using length of culvert in feet as abscissa against discharge in cubic feet per second for a head of 1 foot on the culvert as ordinate.

This diagram is used as follows: Let us assume we desire the size of a vitrified clay pipe culvert 40 feet long when 113 second-feet is to be carried by the culvert. From the highway construction data it is determined that 3 feet is the maximum safe head to use on the culvert. Find 113 second-feet on the abscissa scale. Run up the diagram on a vertical above 113 second-feet to the diagonal line representing a head of 3 feet. From this point run a horizontal line over to the vertical line through the length of the culvert to be used. It is found that a 42-inch clay pipe is required to discharge the required capacity.

Figures 13, 14, and 15 have been compiled for the purpose of showing the equivalent size of pipe culverts when discharging the same quantity of water, the pipe acting under a head of 1 foot.

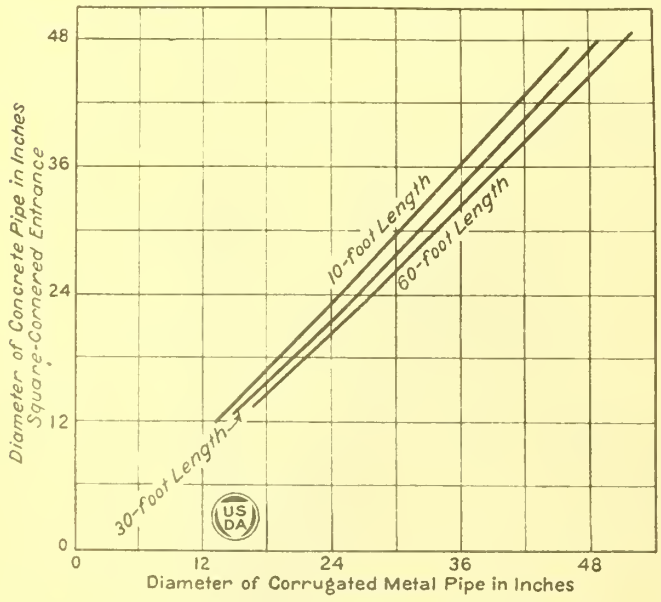

FIG. 14.—Diagram for concrete pipe culverts with square-cornered entrance and corrugated metal pipe culverts showing equivalent sizes when discharging the same quantity. Pipe acting under a head of 1 foot—straight end wall entrance.

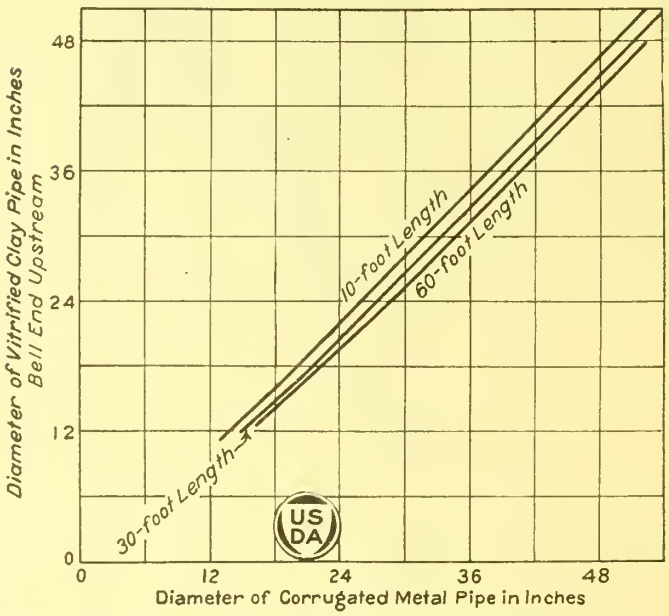

FIG. 15.—Diagram for vitrified clay pipe culverts with bell end upstream and corrugated metal pipe culverts showing equivalent sizes when discharging the same quantity. Pipe acting under a head of 1 foot—straight end wall entrance.

CONCLUSIONS.

1. The discharging capacity of a pipe culvert depends primarily upon the cross section of the pipe and the difference in water level at the two ends of the culvert.

2. To obtain the maximum discharge, the pipe must be so laid as to insure the full cross section of the pipe being filled by the flowing water.

3. If the pipe is laid at too high an elevation with respect to the water levels at the two ends of the culvert, it will not run full and hence will not attain its maximum capacity.

4. If a culvert pipe is so laid that both its upstream and downstream ends are completely submerged the amount of water which it discharges will be proportional to the square root of the difference in water level at the two ends, and the exact grade at which the culvert pipe is laid has no effect whatever upon its maximum discharging capacity.

5. The coefficient of roughness "N" in the Kutter formula for concrete pipe ranges from 0.012 for the 12-inch size to 0.013 for the 30-inch size.

6. The coefficient of roughness "N" in the Kutter formula for vitrified clay pipe ranges from 0.010 for the 12-inch size to 0.013 for the 30-inch size.

7. The coefficient of roughness "N" in the Kutter formula for corrugated metal pipe ranges from 0.019 for the 12-inch size to 0.023 for the 30-inch size.

8. In concrete, vitrified clay, and corrugated metal pipe culverts, 30.6 feet long, with straight end wall entrances:

(*a*) The 12-inch concrete pipe with beveled-lip end upstream will carry about 49 per cent more water than the 12-inch metal pipe.

(*b*) The 18-inch concrete pipe with beveled-lip end upstream will carry about 40 per cent more water than the 18-inch metal pipe.

(*c*) The 24-inch concrete pipe with beveled-lip end upstream will carry about 36 per cent more water than the 24-inch metal pipe.

(*d*) The 30-inch concrete pipe with beveled-lip end upstream will carry about 32 per cent more water than the 30-inch metal pipe.

(*e*) The 12-inch clay pipe will carry about 65 per cent more water than the 12-inch metal pipe.

(*f*) The 18-inch clay pipe will carry about 50 per cent more water than the 18-inch metal pipe.

(*g*) The 24-inch clay pipe will carry about 40 per cent more water than the 24-inch metal pipe.

(*h*) The 30-inch clay pipe will carry about 30 per cent more water than the 30-inch metal pipe.

9. In concrete pipe culverts, 30.6 feet long, with straight end wall entrances:

(*a*) The 12-inch pipe with beveled-lip end upstream will carry about 5 per cent more water than the same pipe with a square-cornered entrance.

(*b*) The 18-inch pipe with beveled-lip end upstream will carry about 9 per cent more water than the same pipe with a square-cornered entrance.

(*c*) The 24-inch pipe with beveled-lip end upstream will carry about 12 per cent more water than the same pipe with a square-cornered entrance.

(*d*) The 30-inch pipe with beveled-lip end upstream will carry about 14 per cent more water than the same pipe with a square-cornered entrance.

10. The effect of the length of the culvert on the discharge is not as great with concrete and vitrified clay pipe as with corrugated metal pipe.

11. The 45° wing walls used in connection with a corrugated metal pipe culvert will increase the capacity from 1 to 10 per cent over that obtained in a metal pipe culvert with a straight end wall.

12. The 45° wing walls used in connection with a corrugated metal pipe culvert are more efficient when set flush with the edge of the pipe than when set 6 inches back from the edge of the pipe.

13. The 45° wing walls used in connection with a corrugated metal pipe culvert are more efficient when built full height to the top of the head wall than when constructed standard height.

14. The 45° wing walls used in connection with a vitrified clay pipe culvert are not as effective as the straight-end wall entrance with the regular bell end upstream.

15. The U-type wings used in connection with a vitrified clay pipe culvert are not as effective as the straight end wall.

16. The beveled-lip end at the entrance of a concrete pipe culvert is a great aid in reducing the entrance loss, especially in the larger sizes.

17. The bell end at the entrance of vitrified clay pipe culvert by virtue of its shape assists greatly in reducing the entrance loss, especially in the smaller sizes.

18. A 24-inch clay pipe culvert, 38 feet long, with a straight end wall and the regular bell end upstream will carry about 10 per cent more water than the same culvert with a square-cornered entrance.

19. By merely rounding the entrance to a 24-inch vitrified clay pipe culvert the capacity may be increased approximately 13 per cent over that obtained with a square-cornered entrance.

20. By projecting the pipe through the head wall so as to obtain the effect of no head wall on the discharge, it was found that—

(*a*) In the 12-inch concrete pipe culvert having a square-cornered entrance, there was little difference whether the pipe projected 3 inches, 2 feet, or 4 feet beyond the head wall.

(*b*) The discharge is decreased slightly by projecting the pipe through the head wall when compared to the same culvert with a straight end wall entrance.

(*c*) The 18-inch corrugated metal with a 3-inch projection beyond the head wall carried slightly more water than the same pipe with either a 2-foot or a 4-foot projection.

(*d*) The 18-inch metal pipe with a straight end wall entrance will carry more water than the same pipe with any length of projection.

21. By increasing the area of the cross section of the outlet end of a pipe culvert so that the area ratio is about 1 to 2, and the angle of divergence about 10°, the discharge of the culvert, when the outlet is submerged, may be increased 40 per cent over that obtained in the same culvert having a uniform bore throughout.

22. The average exponent of H, the head on the pipe culvert, for tables 12 to 69, inclusive, is 0.488. In hydraulics the general practice is to assume that the discharge varies as the square root of the head.

23. New discharge formulas for the flow of water through concrete, vitrified clay, and corrugated metal culvert pipe have been derived from the experimental data. All of these formulas include the loss due to friction in the culvert, the entrance loss, and the velocity head loss. The formulas as derived for culverts 30.6 feet long with straight end wall entrances are as follows:

Concrete pipe with beveled lip end upstream:

$$Q = 4.61\ D^{2.18}\ H^{0.50} \quad . \quad . \quad . \quad . \quad (1)$$

Concrete pipe with square-cornered entrance:

$$Q = 4.40\ D^{2.09}\ H^{0.50} \quad . \quad . \quad . \quad . \quad (2)$$

Vitrified clay pipe with bell end upstream:

$$Q = 5.07\ D^{2.05}\ H^{0.50} \quad . \quad . \quad . \quad . \quad (3)$$

Corrugated metal pipe:

$$Q = 3.10\ D^{2.51}\ H^{0.50} \quad . \quad . \quad . \quad . \quad (4)$$

ROAD MATERIAL TESTS AND INSPECTION NEWS.

PUBLIC ROADS will hereafter contain a department which will be of interest primarily to testing and materials engineers, and which will contain notes, comments, and items of general interest in the field of testing and inspection of road materials.

It has been felt for some time that the laboratories of the bureau should keep in closer touch with various State and university testing laboratories and material departments, and it is believed that the publication of items of interest in this field in a special department of PUBLIC ROADS will contribute materially toward this end. The information will take the form of short concise articles dealing with: (1) Special methods of testing which are being used by either the Bureau of Public Roads or the State laboratories; (2) unusual or interesting materials or combinations of materials proposed for use; (3) apparatus used for testing road materials, with special reference to new designs or modified forms, as well as precautions to be observed in connection with its calibration; and (4) the results of cooperative check tests made from time to time by the various State, university, and commercial laboratories.

Testing and materials engineers, chemists, and all others engaged in the testing and inspection of road materials and in highway research are invited to submit articles for publication in this department of PUBLIC ROADS. These contributions may cover any matter of interest in connection with the subjects outlined above. It is believed that there must be an immense amount of information in the possession of the various testing laboratories which would be of interest to others working along the same line. Needless to say, due credit will be given in all cases. Contributions should be addressed to the U. S. Bureau of Public Roads, Washington, D. C., attention Mr. A. T. Goldbeck, Chief, Division of Tests.

COMPARATIVE RESULTS OF CHECK TESTS Sd–4.

On November 8, 1923, the bureau distributed to the various cooperating laboratories mimeographed copies of check tests on a sample of concrete sand, identified as Sd–4. There are a number of points in connection with this check test which are of interest.

In the first place, it will be observed that, whereas most of the laboratories use the so-called American Society for Testing Materials standard sieves for making the sieve analysis, a number of laboratories are using the Tyler standard. The bureau is not prepared to go into a discussion of the relative merits of these two series at this time, except to say that there would appear to be no good reason why some generally accepted standard should not be adopted in the near future. In this connection it might be said that a subcommittee of the Committee on Methods of Tests of the American Society for Testing Materials has recently been organized for the purpose of considering the various standard screen scales for testing sieves, with the object of adopting a standard for general use. Any suggestions relative to this proposed new standard will be welcomed by the committee, and should be forwarded to the chairman, Mr. F. G. Breyer, New Jersey Zinc Co., Palmerton, Pa.

Another point of interest in connection with this test is the fact that numerous laboratories are using samples for the sieve analysis in excess of the weight specified by the Committee on Tests and Investigations of the American Association of State Highway Officials, as published in U. S. Department of Agriculture Bulletin 949. The test as given in this bulletin (Test No. 10, page 12) specifies that the sample shall weigh not less than 100 nor more than 500 grams. Likewise, in connection with the test for silt content, reference to Bulletin 949, test No. 12, shows that the sample for this test should weigh 500 grams. The mimeographed results indicate that a number of laboratories did not use the specified weight. One of the most serious things in connection with this check test, however, is the wide variation in the results of the tensile strength ratio. This may be due to a number of causes. It is believed, however, that the primary reason is the lack of a standard method of estimating normal consistency of the test sand mortar. Variations in the quality of cement used may likewise play a part, as well as variations in methods of manipulation. It is evident that further work must be done on the standardization of this test if the laboratory men expect it to be taken seriously. A review of the temperatures reported by the various laboratories is also interesting in showing that the average temperatures are in all cases above the 21° C. specified as the standard testing temperature for cement.

COMPARATIVE RESULTS OF CHECK TESTS Ct–4.

Several reports have been received of tests made on the check sample of cement sent out to the various laboratories on October 28, 1923. Reports of fineness show great variations in the results obtained. In this connection, the attention of the bureau has been called by the U. S. Bureau of Standards to the fact that 200-mesh testing sieves as now sent out by manufacturers are apt to be far from accurate. It has recently been almost impossible for the Bureau of Standards to secure 200-mesh testing sieves which conform to its requirements regarding size of opening. The attention of the various laboratories is called to the fact that the Bureau of Standards is prepared to calibrate testing sieves for a nominal charge, and anyone who has the slightest doubt as to the accuracy of his sieve is urged to avail himself of this opportunity. In the event that the sieve can not be spared long enough to be sent to Washington for calibration, the Bureau of Standards has arranged to send out standard samples of cement which may be tested on the sieve in question and its accuracy thereby established. The price of these standard samples is 50 cents each.

ASH CORRECTION IN DETERMINATION OF PER CENT BITUMEN.

In examining check test reports on samples of bituminous aggregates it has been noted that in some cases high results for per cent bitumen have been obtained. It is believed that in some instances this may have been caused by not taking cognizance of the fact that in making the tests for per cent bitumen some finely divided mineral matter is almost invariably to be found in the extract. Unless a correction (commonly designated as ash correction) is made for this mineral matter, the results for percentage of bitumen will be high, due to the fact that this mineral matter will be included with the bitumen. Methods for determining this mineral matter are described on page 57 of U. S. Department of Agriculture Bulletin 949.

SUBGRADE STUDIES.

The research work of the subgrade laboratory of the Bureau of Public Roads has progressed to a point which has necessitated the development of several new tests before some of the important characteristics and properties of soils could be properly detected and analyzed. These characteristics are in addition to the properties ordinarily determined by drying, by mechanical analysis, and by slaking. It is necessary to determine the bearing power of different soils as affected by area of bearing block and the percentage of capillary water present in the soil. The amount of silt, clay, and ultraclay must be determined. The water equivalent and the action of the soil under frost conditions is also important. All of these properties and characteristics must be determined before a soil may be properly classified and its action under subgrade conditions predicted. This work has lead to some very important indications relative to the proper treatment of a particular subgrade soil to improve its stability and bearing power. For instance, among other interesting facts, it has been determined that a blanket of sand or other suitable granular material on a subgrade which is more or less continuously wet and soft will prevent this material from working up into a stone subbase or through the voids of the stones forming a macadam road, thus preventing the pumping action so often seen in connection with this type of road. The work so far has been mainly in connection with laboratory and local field conditions, but is now being extended into field work in various localities of the United States. This work will enable the bureau to properly analyze and classify the soils of different sections of the country and to prescribe proper treatment of the subgrade in any particular case to insure its improvement.

PENETRATION LIMITS FOR ASPHALT.

At a conference called by the Division of Simplified Practice, U. S. Department of Commerce, the following penetration limits were adopted and were recommended to become effective on all deliveries of material after January 1, 1924.

For construction of sheet asphalt, asphaltic concrete, and asphalt macadam pavements, and also for surface treatment—Penetration limits.

25 to 30	50 to 60	100 to 120
30 to 40	60 to 70	120 to 150
40 to 50	85 to 100	150 to 200

For joint filler for various types of construction—Penetration limits.

30 to 50	60 to 70
50 to 60	85 to 100

CEMENT TESTING.

Among the much-abused requirements found in the specifications for testing Portland cement is one which reads as follows: "The quantity of dry material to be mixed at one time shall not exceed 1,000 grams nor be less than 500 grams." It is astonishing to observe the false economy practiced in some laboratories in mixing batches for normal consistency or pats. Quantities as small as 50 grams have been mixed and it often happens that 100 and 200 gram batches are used. When it is realized that considerable evaporation and loss of moisture on the mixing table occurs during the manipulation of cement paste and mortar, the necessity for larger batches will be seen. The objection to working more than 1,000 grams at a time is that there is a tendency to undermix the mortar, with consequent production of nonuniform specimens of lowered strength.

NEED FOR A CRUSHED STONE ABRASION TEST.

The value of standardizing some form of stone test which can be applied to the crushed product so as to make car sampling and car testing practicable has been recognized for some time, but so far, apparently, efforts to standardize such a test made have proved unsuccessful. The Bureau of Public Roads has been experimenting for some time in the hope of developing such a test. Further details regarding the method of testing tentatively adopted, as well as some results of a cooperative series of tests made by several laboratories on a sample of stone using this method, will be given in the next News Letter.

○

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS.

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets, nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

REPORTS.

Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.
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*Report of the Chief of the Bureau of Public Roads for 1922. 5c.
*Report of the Chief of the Bureau of Public Roads for 1923. 5c.

DEPARTMENT BULLETINS.

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220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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*555. Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials. 10c.
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161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

Public Roads Vol. III, No. 25. Automobile Registrations, Licenses, and Revenues in the United States, 1919.
Vol. III, No. 29. State Highway mileage, 1919.
Vol. III, No. 36. Automobile Registrations, Licenses, and Revenues in the United States, 1920.
Vol. IV, No. 5. Automobile Registrations, January 1 to July 1, 1921.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH.

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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I BELIEVE that highway transportation is a necessary and integral part of this connecting stone in civilization's arch and is coequal with other forms of transportation in sustaining the body of the structure.

I BELIEVE that my mission, as a highway engineer, is to assist in shaping and improving the highways of my country, in harmony with those who provide the vehicles which are their necessary complement, to the end that, joined with other means of transportation, they may meet the need of our people for easy, quick, and untrammelled transportation.

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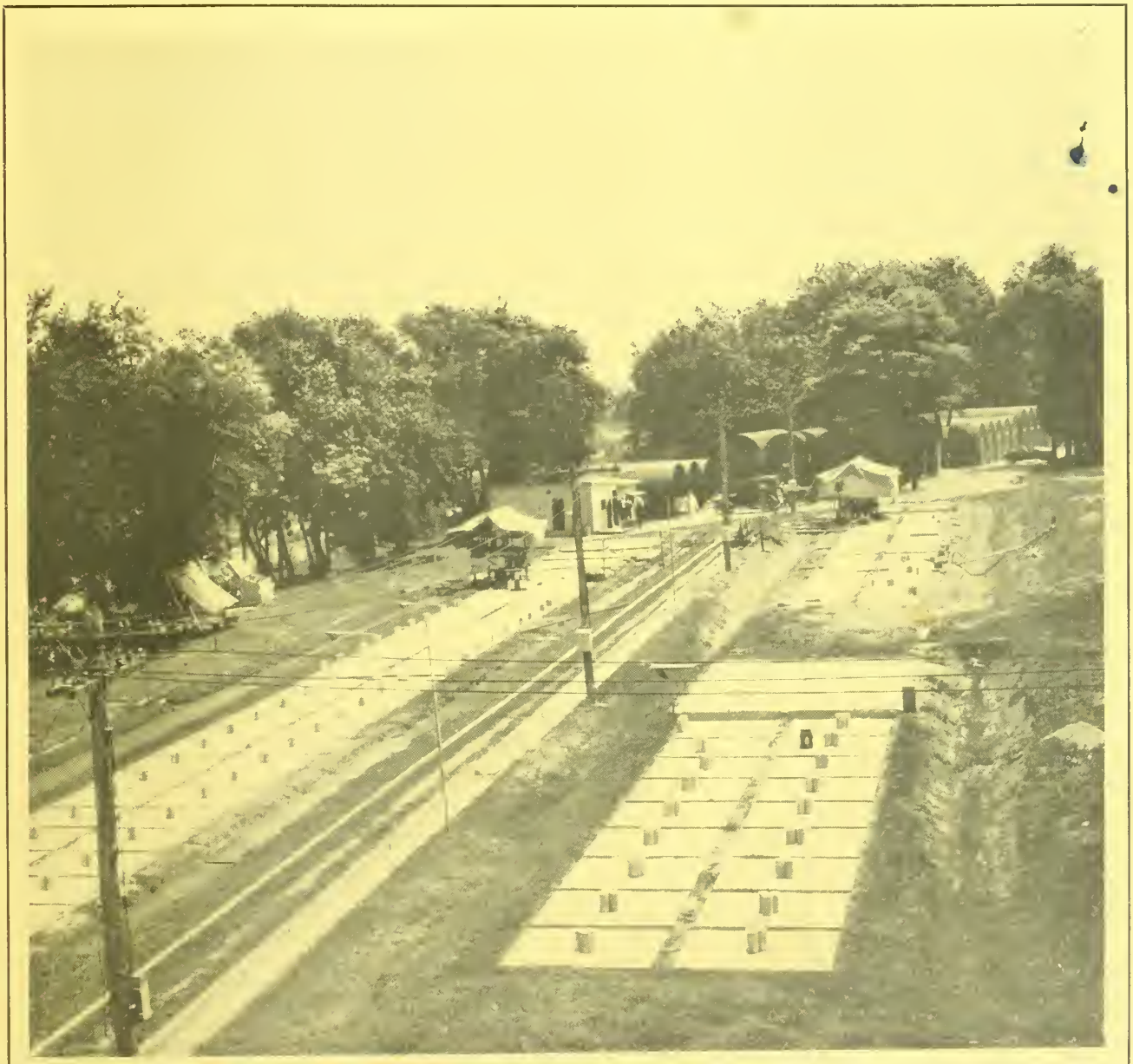

GENERAL VIEW OF SLABS FOR IMPACT TEST, ARLINGTON, VA.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

VOL. 5, NO. 2 APRIL, 1924 H. S. FAIRBANK, Editor

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IMPACT TESTS ON CONCRETE PAVEMENT SLABS.

RESISTANCE OF VARIOUS DESIGNS DETERMINED BY BUREAU OF PUBLIC ROADS EXPERIMENTS.

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CONTINUING the study of the magnitude and effect of motor truck impact begun four years ago, the Bureau of Public Roads has recently completed a series of tests to determine the resistance of pavement slabs of various designs to impact of motor vehicles. Previous reports dealing with earlier phases of the investigation have announced the findings of the bureau with respect to the possible magnitude of the impact forces resulting from the operation of trucks over road surfaces of different degrees of roughness, the relative strength of such forces compared with the static weight of the vehicle, the effect of differences in tire and spring equipment and variations in sprung and unsprung load upon the character and intensity of the impact, and the effect of the impact upon a limited number of specially constructed slabs of various designs.[1]

This report deals with the results of the tests of a second series of slabs more comprehensive in its range of types. As in the case of the previous reports, this one deals with only one phase of the investigation which is being continued. The results reported are not entirely conclusive. They are to be regarded as sign boards which point the way rather than as a guaranty of safe arrival at the destination. If viewed thus broadly, it is felt that their publication will accomplish a useful purpose.

Some of the results are erratic and in the absence of check tests can not be explained, but in the main they are quite consistent. Some of the indications are apparent throughout the tests, while others do not show so clearly in the data, and in such cases careful observations made in the course of the tests and the knowledge of the investigators as to conditions which influenced the results of individual tests have been drawn upon as well as the recorded test data in forming the conclusions which are presented below.

CONCLUSIONS DRAWN FROM THE TESTS.

The exact nature of the tests must be borne in mind in applying the conclusions drawn from them to other conditions. The conclusions are stated in terms of maximum impacts delivered by the different weights of motor truck. Failure is assumed to have occurred when cracking takes place. The maximum impact values are such as would be expected under conditions of high speed and rough surface. Smoothness of the surface and suitable tire equipment will reduce the maximum impact pressures with a resulting higher load carrying capacity than that indicated by the tests.

The essential features of the tests involve:

(1) Specimens 7 feet square laid on a moderately plastic clay subgrade, the wet portion having water standing almost level with the surface, the dry portion being merely damp with capillary moisture.

(2) Repeated impacts increasing to the maximum which caused cracking applied at the corner and at the center of one edge by an impact machine dealing blows closely resembling impact of motor trucks. Rolling loads were not applied. No impacts were delivered at the centers of the slabs.

Construction view—wet subgrade section showing tile cross drains and subgrade cut for slabs of various thickness.

The conclusions drawn from the test are as follows:

The resistance of the road slab depends in part upon the supporting value of the subgrade. A subgrade of high supporting value materially increases the resistance to impact.

Impact resistance of rigid slabs varies neither directly as the depth of the slab nor as the square of the depth but as some power less than two.

In general, plain concrete slabs show no more resistance to impact delivered at the edge than to impact delivered at a corner.

Transverse cracks and longitudinal cracks near the sides of a road slab may be caused by impact delivered at the edge of the slab.

Plain concrete of 1:3:6 mix offers resistance to impact ranging from about 60 per cent to 80 per cent of the resistance of plain concrete of 1:1½:3 mix. The lean mix also shows more variation in strength.

Reinforcing steel in concrete slabs, if present in sufficient amount and so placed as to receive tensile stress, adds to the resistance of the slab to impact.

Reinforcing steel placed longitudinally and transversely in equal percentages is more effective in preventing corner failures than the same amount placed in one direction.

For a given percentage of steel, small deformed rods closely spaced seem to be more effective than large deformed rods widely spaced.

There is very little evidence of cushioning by bituminous tops on concrete bases at temperatures of 90° F. or less.

In these tests there was no evidence that bituminous tops on concrete bases added to the slab strength of the base, with the possible exception of the 4-inch and 6-inch bases on the dry subgrade.

Laid on the wet plastic subgrade none of the unreinforced slabs was capable of resisting impact at the edge or corner equivalent to that of a 5-ton truck; only the 8-inch, 1:1½:3 slabs and the 2-inch Topeka

[1] Previous reports on these investigations will be found in several issues of Public Roads, as follows: Vol. 3, No. 35, March, 1921; Vol. 4, No. 6, October, 1921; Vol. 4, No. 7, November, 1921; and Vol. 4, No. 8, December, 1921.

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tops on 8-inch, 1:1½:3 bases resisted edge or corner impact equivalent to that of a 3-ton truck; all unreinforced slabs of lesser thickness failed under edge or corner impact less than that of a 2-ton truck.

Laid on a dry subgrade the 8-inch plain concrete slabs of 1:1½:3 mix and the 8-inch edge thickness, unreinforced, 1:1½:3 bases with 2-inch Topeka tops resisted edge and corner impact equivalent to that of a 5-ton truck with a safe margin; no other slabs were capable of resisting the 5-ton truck impact even under the favorable conditions of dry subgrade support. A section of 6-inch plain concrete base, 1:1½:3 mix, with a 2-inch Topeka top resisted the edge impact of a 3-ton truck; no other slabs of lesser edge thickness, laid on the dry subgrade, were capable of resisting impact greater than that of a 2-ton truck.

None of the systems of steel reinforcing tested added sufficiently to the strength of a 6-inch 1:1½:3 concrete slab to enable it to resist the edge and corner impact of a 3-ton truck when the slab was supported by a very wet, plastic subgrade, nor to resist the edge or corner impact of a 5-ton truck when the slab was supported by a dry subgrade.

In some of the tests there was evidence that while the presence of the steel did not assist greatly in preventing the formation of the first crack it did prevent the development of the crack and the further failure of the slab.

OBJECT AND ORDER OF THE TESTS.

The object of this series of tests which, like the other series, was conducted at the Arlington Experimental Farm, Arlington, Va., was to secure data on the comparative resistance of sections of various types of road slabs when subjected to impact forces of the same magnitude as those applied to actual roads by modern truck traffic of different weights.

To carry out this purpose three distinct steps were required, as follows:

1. The determination of the magnitude of the maximum impact to which a road slab is subjected by motor trucks of different sizes.
2. The application of similar impact forces to the test sections.
3. The measurement of the effect of these impacts on the various test sections.

In order to carry out the first step in the program, a series of about 100 road tests was run to determine the maximum impact to which a road slab is subjected when different sizes of motor trucks are driven over it, fully loaded and at their maximum speed.

At the present time, the Bureau of Public Roads is conducting a similar but much more elaborate series of tests to determine this and other information regarding actual motor truck impact. But for the purpose of the impact tests on pavement slabs the tests were limited to standard types of 2-ton, 3-ton, and 5-ton trucks. Standard equipment in the way of solid tires, wheels, springs, and spring suspension was used. These trucks were driven over typical concrete roads, maintained in good condition, and the maximum rear wheel impact was measured by means of an especially designed accelerometer.

DESCRIPTION OF THE TEST SECTIONS.

As in the first series of impact tests, the attempt was made to reproduce two subgrade conditions—a dry, well-drained type and a thoroughly saturated one. Extra precautions were taken to insure as much contrast as possible between the bearing values of the two subgrades.

One hundred and twenty-four slabs were laid, embracing in all about 40 different types. Each slab was laid in duplicate, and the more important types were placed on both the dry and the wet subgrades. All slabs were 7 feet square.

The types of slabs may be roughly divided into five groups:

1. Plain concrete.
2. Reinforced concrete.
3. Concrete bases and bituminous tops.
4. Bituminous bases and bituminous tops.
5. Macadam bases and bituminous tops.

Tests of the slabs of all five groups have been completed, but this report does not deal with the tests of bituminous tops on macadam and bituminous bases. The behavior of these slabs is materially different from that of the concrete slabs and the concrete bases with bituminous tops, and it is felt that the results of the tests of these groups should receive further study before they are announced.

A detailed description of each of the slabs in groups 1, 2, and 3 as to materials used, mixtures and thickness, is given in Table I.

TABLE I.—*Description of test slabs.*

(Series 222–267 laid on wet subgrade; Series 314–337 laid on dry subgrade; Series 1R–18R laid on dry subgrade.)

PLAIN CONCRETE SLABS AND BASES.

Slab No.	Base course.		Binder course.		Surface course.	
	Thickness.	Material.	Thickness.	Material.	Thickness.	Material.
	Inches.		*Inches.*		*Inches.*	
222	6	1:3:6 concrete.	1½	Bituminous concrete.	1½	Sheet asphalt.
223	6	do	1½	do	1½	Do.
224	6	do			4	Bituminous concrete.
225	6	do			4	Do.
226	6	do			2	Do.
227	6	do			2	Do.
228	4	do			2	Topeka.
229	4	do			2	Do.
230	4	1 : 1½ : 3 concrete.			2	Do.
231	4	do			2	Do.
232	6	1:3:6 concrete.			2	Do.
233	6	do			2	Do.
234	6	1 : 1½ : 3 concrete.			2	Do.
235	6	do			2	Do.
236	8	1:3:6 concrete.			2	Do.
237	8	do			2	Do.
238	8	1 : 1½ : 3 concrete.			2	Do.
239	8	do			2	Do.
240					4	1 : 1½ : 3 concrete.
241					4	Do.
242					6	Do.
243					6	Do.
244					6	1 : 3 : 6 concrete.
245					6	Do.
246					8	1 : 1½ : 3 concrete.
247					8	Do.
248					8	1 : 3 : 6 concrete.
249					8	Do.
314	6	1 : 1½ : 3 concrete.			2	Topeka.
315	6	do			2	Do.
316	6	1:3:6 con-			2	Do.
317	6	do			2	Do.
318	4	1 : 1½ : 3 concrete.			2	Do.
319	4	do			2	Do.
320	8	do			2	Do.
321	8	do			2	Do.
322					4	1 : 1½ : 3 concrete.
323					4	Do.
324					6	Do.
325					6	Do.
326					6	1 : 3 : 6 concrete.
327					6	Do.
328					8	1 : 1½ : 3 concrete.
329					8	Do.

TABLE I.—*Description of test slabs*—Continued.

CONCRETE SLABS WITH MESH REINFORCING—WET SUBGRADE.

Slab No.	Surface course. Thickness.	Material.	Reinforcement. Description.	Per cent.
	Inches.			
250	4	1: 1½: 3 concrete	1 layer No. 6	0.21
251	4	do	do	.21
252	4	do	2 layers No. 6	.42
253	4	do	do	.42
254	4	do	2 layers No. 8	.66
255	4	do	do	.66
256	4	do	1 layer No. 10	.41
257	4	do	do	.41
258	4	do	2 layers No. 10	.82
259	4	do	do	.82
260	6	do	1 layer No. 6	.19
261	6	do	do	.19
262	6	do	2 layers No. 6	.38
263	6	do	do	.38
264	6	do	2 layers No. 8	.46
265	6	do	do	.46
266	6	do	2 layers No. 10	.58
267	6	do	do	.58

CONCRETE SLABS WITH MESH REINFORCING—DRY SUBGRADE.

Slab No.	Surface course. Thickness.	Material.	Reinforcement. Description.	Per cent.
	Inches.			
330	4	1: 1½: 3 concrete	1 layer No. 6	0.21
331	4	do	do	.21
332	4	do	1 layer No. 10	.41
333	4	do	do	.41
334	6	do	2 layers No. 6	.38
335	6	do	do	.38
336	6	do	2 layers No. 8	.46
337	6	do	do	.46

CONCRETE SLABS WITH ROD REINFORCING—DRY SUBGRADE.

Slab No.	Surface course. Thickness.	Material.	Reinforcement. Number of layers.	Size of rods.	Spacing of rods center to center.	Per cent.
	Inches.			*Inch.*	*Inches.*	
1-R	6	1: 1½: 3 concrete	1	⅜	3¾	0.5
2-R	6	do	1	⅜	3¾	.5
3-R	6	do	2	⅜	3¾	1.0
4-R	6	do	2	⅜	3¾	1.0
5-R	6	do	2	⅜	7½	.5
6-R	6	do	2	⅜	7½	.5
7-R	6	do	1	½	10½	.5
8-R	6	do	1	½	10½	.5
9-R	6	do	2	½	10½	1.0
10-R	6	do	2	½	10½	1.0
11-R	6	do	2	½	20½	.5
12-R	6	do	2	½	20½	.5
13-R	6	do	1	⅜	20	.5
14-R	6	do	1	½	20	.5
15-R	6	do	2	⅜	20	1.0
16-R	6	do	2	½	20	1.0
17-R	6	do	2	⅜	40	.5
18-R	6	do	2	½	40	.5

CONSTRUCTION OF THE TEST SECTIONS.

Every effort was made to have each slab built exactly according to specifications. The subgrade on which the slabs rest was brought to grade by cutting. No fills were permitted. In the wet subgrade, a tile drain was placed transversely, every eight feet, connecting the longitudinal side ditches. These tile lines came between slabs so as not to disturb the soil under the slabs and their function was to lead water from the side ditches back into the subgrade.

All concrete was mixed in a standard gasoline-driven mixer, the materials being measured in a cubic-foot measure. The coarse aggregate used was washed,

Construction view—wet subgrade section, bases laid and forms for bituminous material in place.

Potomac River gravel and the fine aggregate a good quality of concrete sand purchased locally. As soon as the concrete was poured it was protected with canvas until the following morning when it was covered with earth and kept wet for two weeks. Curing was completed by exposure to the air.

Three 6 by 12 inch compression cylinders were made of the concrete as it came from the mixer and these were cured in the same manner as the slab.

All concrete was placed rather dry and was finished by striking off flush with the forms with a screed. No trowcling was permitted.

The plans called for all steel in the reinforced slabs to be placed 2 inches from the top surface of the slabs. For the reinforcing itself, except in the rod-reinforced sections, a fabric type was used—rods one way 6 inches apart, held together with perpendicular wire cross members every 12 inches. The cross members were not welded to the rods, but were attached to them with wire clips. Greater bond strength would probably have been developed had these members been electrically welded. When two layers of reinforcing were used, the rods were placed at right angles to each other with one sheet laid directly on the other. Test on this steel showed it to be an intermediate grade steel with an ultimate strength of 70,000 pounds. In sections 1–R to 18–R, square, deformed bars were used, the sizes and spacing being given in Table I.

After the concrete had cured for one year, the bituminous slabs were laid. This was done by contract, the material being mixed and placed under the supervision of the Bureau of Public Roads.

Four and six inch concrete bases on the wet-subgrade sections were cracked in rolling the bituminous tops, the lean mixes being badly broken up. This unfortunate occurrence eliminated several very interesting sections from the tests, but it is worth noting that 6-inch 1 : 1½ : 3 concrete bases were cracked in construction where a fairly bad subgrade condition obtained, while 4 inch bases of the same mix withstood the same roller loads undamaged on a good subgrade. While the ditches surrounding the wet-subgrade sections had not been filled with water up to the time the rolling took place, the position of these sections at the foot of the hill and the presence of capillary water created a naturally poorer subgrade condition than that which existed in the dry-subgrade sections.

TESTING THE SLABS.

The second step in the program called for the application of impact to the test sections. The testing was begun in May, 1923. Prior to this time the ditches surrounding the wet subgrades had for several months been kept filled with water to the level of the under side of the 6 inch slabs. The magnitude of the impact forces to be applied had previously been determined by the preliminary road tests using actual motor trucks. As it was obviously impracticable to use a motor truck for testing the slabs, a special portable impact machine was designed and two of them were built.

THE IMPACT MACHINE.

This machine, which is shown in the accompanying illustration, consists essentially of a rear wheel of a motor truck fastened solidly to the middle of a truck spring and so arranged that it can be raised to any desired height from the surface under test and dropped suddenly. Power is furnished by an electric motor which operates through a train of gears, raising the wheel by means of a pair of cams. The whole mechanism is suitably mounted on a framework of structural steel.

Adjustments permit changing tire, wheel, spring, sprung weight, unsprung weight, or height of drop at will. The impact machine proved to be very satisfactory and reliable and no trouble was experienced from this source throughout the tests.

Load conditions corresponding to those of 2-ton, 3-ton, and 5-ton trucks were used.

THE METHOD OF TEST.

In testing a slab the machine was set up so that the wheel was over the point where the load was to be applied. The unsprung weight was made to correspond to that of a 2-ton truck. By means of the corner screws the machine was next set for a very low drop, usually one-quarter inch or less. The truck spring was then loaded to equal the sprung load of the same truck. This was done by pulling down the ends of the spring with screws until its deflection indicated the required sprung load. From three to five blows were delivered, after which the machine was raised slightly and three to five blows were again delivered at the new height. Each time the machine was raised it was necessary to pull the ends of the spring down an equal amount so that the spring deflection would remain constant. This procedure was continued until the data indicated that the maximum impact of which this weight of truck was capable had been delivered. If the slab had not failed, the loadings were increased to those of a 3-ton truck and corresponding impacts delivered. If a 5-ton truck impact failed to break the slab, the impacts were increased until failure occurred.

POINTS OF LOADING USED.

As the data desired were only comparative, it was originally intended to test one of each pair of slabs at the center point and the other one at the corner. But road tests indicated that the edge of a slab was one of its weakest points. Also, the testing of the first series of slabs had shown that for small slabs center resistances are dependent upon the area of the slab.[1] With these points in mind, it seemed that more and better information could be obtained by testing not at the center of the slab but at the center of the edge. The results show this to be true. These are called edge tests in the data.

Rolling asphalt tops. Roller weight, 250 pounds per inch of width.

In the corner tests the load was applied as near to the corner as it was possible to get the wheel and still have the area of contact entirely on the slab. This means the center of the wheel was over a point which was about 7 inches back from the corner, measuring along the diagonal of the slab.

DATA OBTAINED.

To compare the various slabs it was necessary to know the maximum impact applied to the slab by the wheel as it comes down and strikes the surface. There are several ways of doing this, all of which have been used at different times in connection with these tests. Among these methods may be mentioned: (1) A space-time curve; (2) copper-cylinder method; (3) deformation of spherical steel surface (Kreuger apparatus); and (4) mass $\times$ acceleration method, measuring the acceleration with an accelerometer.[2]

The first method is probably the most precise and the most difficult to use successfully. Because of the time required, it is impracticable for such tests as these. The second method is an approximation, as it introduces a new factor for which correction can not readily be made; i. e., the cushioning effect of the cylinder itself. The third method offers practical difficulties which render it unsuitable for this work.

By far the most satisfactory method is the mass $\times$ acceleration method, using an accelerometer to measure the acceleration. For this purpose a special accelerometer was used, which was devised and developed in the Bureau of Public Roads. This instrument is described in Proceedings of the American Society for Testing Materials, 1923, "An accelerometer for measuring impact," by E. B. Smith. In use it gives a reading which indicates the maximum negative acceleration which occurs when the falling truck wheel is brought to rest by the resistance of the slab. Knowing the unsprung weight of the impact machine, it is a simple matter to calculate the maximum impact by means of the formula

[1] Public Roads, Vol. 4, No. 7, November, 1921. "Tests of impact on pavements by the Bureau of Public Roads."

[2] For descriptions of these methods see Public Roads, Vol. 4, No. 6, October, 1921. "Tests of impact on pavements by the Bureau of Public Roads."

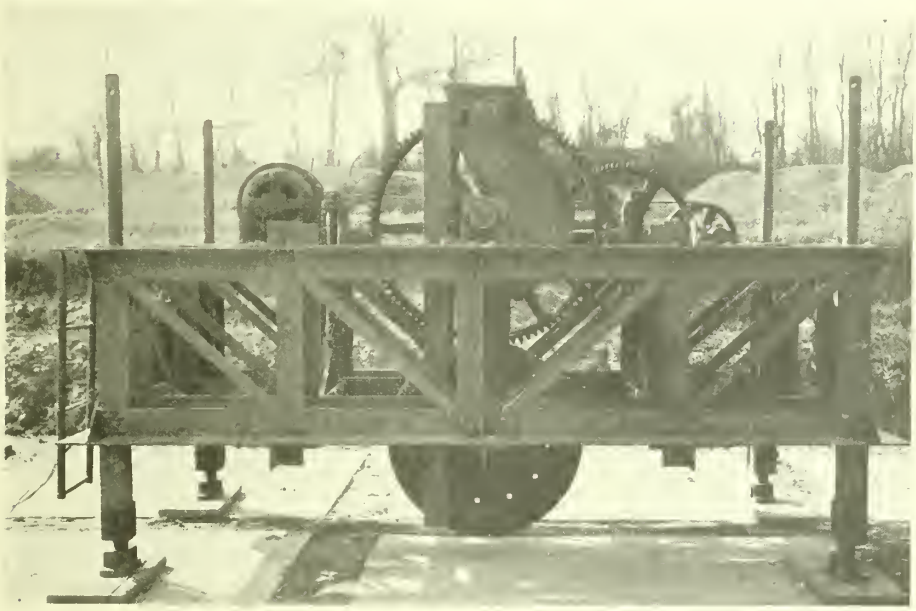
$$F = Ma \div P$$

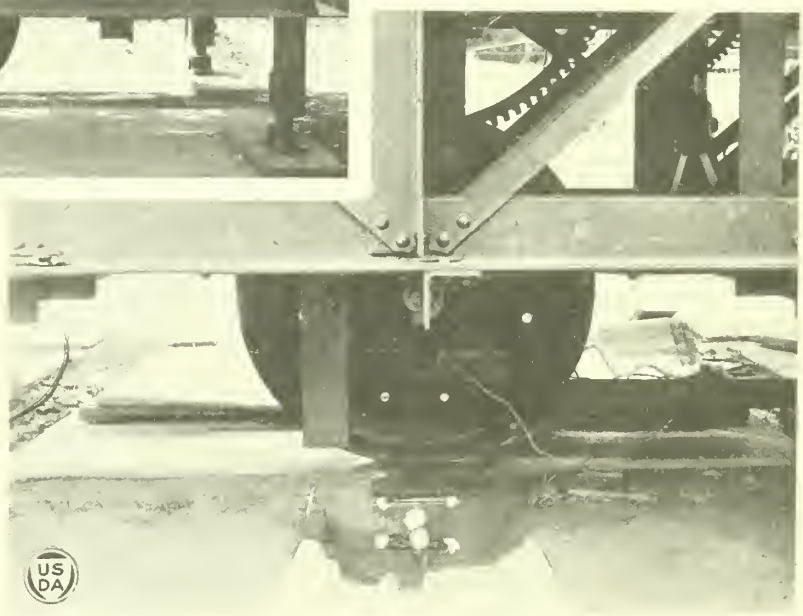
in which F = Maximum force (impact).

$M = \frac{W}{g}$ = weight of unsprung portion divided by 32.2.

P = spring pressure existing when wheel is in contact with slab.

In practice this method has proved to be easy to use and sufficiently accurate for all practical purposes. Inasmuch as the accelerometer was actually mounted on a motor truck in the preliminary road tests, previously described, and certain readings obtained and then later it was mounted on the impact machine where the sprung and unsprung weight conditions of the truck were duplicated, it seems reasonable to assume that impacts which, under these two conditions, give equal accelerometer readings are equal in magnitude.





TOP.—THE IMPACT MACHINE.

BOTTOM.—CLOSE-UP OF IMPACT MACHINE AND SET-UP OF APPARATUS FOR THE EDGE TEST, SHOWING GRAPHIC STRAIN GAUGES IN TOP AND BOTTOM OF THE CONCRETE BASE, DEFLECTION DIAL, THERMOMETER IN BITUMINOUS MATERIAL, AND ACCELEROMETER MOUNTED ON SHELF BELOW AXLE.

The effect of the impact is to deflect the slab. It is necessary, therefore, to measure the amount of this deflection. An Ames dial reading to one-thousandth of an inch and "choked" to hold its maximum reading, was used for this purpose.

This deflection of one part of the slab caused certain stresses of compression and tension to be set up in the material itself. Concrete being weak in tension, the tensile stress is obviously the governing one. Hence it was necessary to measure the maximum tensile stress created by the impact. An instrument known as a graphic strain gage was used very successfully to measure these deformations. This instrument also was devised in the Bureau of Public Roads, and its construction and operation is described in "Engineering News-Record," March 22, 1923, in an article entitled "A new impact strain gage," by A. T. Goldbeck.

In addition to the data thus obtained, it was necessary to know exactly the height of drop used each time. For this purpose a stylus connected directly to the truck wheel was used to trace a record on a paper which was carried on a drum. By determining the point at which the wheel comes in contact with the slab this total height of drop can be divided into the component parts of free fall, rubber deformation and slab deflection.

AUXILIARY TESTS.

In addition to the actual impact test on each slab, an auxiliary test was always made to determine the load supporting value of the subgrade. Also in the case of the concrete slabs, compression tests were run on the 6 by 12 inch test cylinders, previously mentioned, to determine the modulus of elasticity and the crushing strength of the concrete at the time of test.

The subgrade bearing value determination was made with a portable apparatus shown in the accompanying illustration. The method of test is to apply a constantly increasing unit load to a small circular bearing block scraped to intimate contact with the subgrade, and to measure the penetration of this foot into the soil under the load. This test is described in Public Roads, volume 4, No. 5, September, 1921. "Preliminary report on the Bates experimental road."

The determination of the modulus of elasticity by means of a compressometer is familiar to all and will not be described except to say that the instrument devised and built by the Bureau of Public Roads and described in American Society for Testing Materials Proceedings, 1923, "Slag as a concrete aggregate," by Raymond Harsch, gives excellent results and apparently is free from the lag so common in instruments of this type.

Typical modulus of elasticity curves for concrete of the two mixes used are shown in the accompanying diagram.

TABLE II.—*Average compressive strength of all test cylinders.*

Class of sections.	Compressive strength in pounds per square inch.	
	1:3:6 mix.	1:1½:3 mix.
Dry subgrade	1,420	4,180
Dry subgrade, rod-reinforced slabs		5,948
Wet subgrade	1,477	4,459

THE SUBGRADE AND ITS EFFECT ON SLAB RESISTANCE.

These slab tests have especially emphasized the important part the subgrade plays in the resistance of a road slab of any type. The rigid slabs on the wet subgrade showed roughly two-thirds of the resistance of duplicate slabs on the good subgrade.

The results of the subgrade determination varied somewhat, but it was nearly always possible to get two curves to check closely out of three or four tests, which is the number of tests usually made at each slab. The load-penetration curve was plotted for each test and these curves averaged for each subgrade. The three average curves grouped in the figure reproduced on page 10 show clearly the difference in bearing value of the three subgrades. There was considerable variation in moisture content of the samples taken and no relation could be found between moisture content and bearing value. The reason for this is thought to be primarily in the varying soil characteristics of the different samples due to the topographic location of the different slabs. No laboratory analysis was attempted on the many samples taken. Except to say that the moisture for the samples taken ran on the average from 5 to 10 per cent higher on the wet subgrade than on the dry, it is not thought worth while to present the data on these moisture tests.

BEHAVIOR OF THE CONCRETE SLABS.

It was considered that a rigid slab had failed when the strain gage record indicated the formation of the first crack. This was selected as a definite point of comparison for slabs of this type, but the behavior of the section beyond this point was also considered in comparing reinforced types.

It is not possible to derive an exact mathematical relation from the results of such tests as these, but it would seem from the examination of the most nearly comparable sections that the resistance of the slabs varies neither directly with the depth nor with the square of the depth, but as some power less than two. There are other factors present on which there are no data, such as the relation between bending and shearing stresses in different depths of slabs, which may be, and probably are, very important.

Impacts were applied at the corner and at the center of one side of each type of slab. There seems to be practically the same resistance at both points of loading in the plain concrete slabs. When there was steel present in sufficient amount and in the proper place to resist tension, the tests showed a corresponding increase in resistance.

The edge test invariably resulted in a transverse crack beginning with an incipient crack on the bottom of the slab directly under the wheel. This crack developed rapidly across the slab. Thin slabs punched out immediately following the formation of the crack. One 8-inch, 1:3:6 slab developed a longitudinal crack which divided the slab into quarters. It is probable in this case that the inertia of the heavy slab caused tension which exceeded the strength of the concrete.

The corner test caused a curved crack across the corner due to tension in the top fibers of the slab. The unit fiber deformation at failure was the same

TABLE III.—*Test data on rigid slabs.*

LAID ON WET SUBGRADE.

Slab No.	Construction: Concrete base, Thickness	Concrete base, Mix	Top course, Thickness	Top course, Material	Point of application of load	Maximum impact resisted	Maximum unit fiber deformation, Elastic	Maximum unit fiber deformation, At failure	Average modulus of elasticity at failure	Average crushing strength of 3 cylinders
	Inches		*Inches*			*Pounds*			*Pounds per sq. in.*	*Pounds per sq. in.*
222	6	1:3:6	1½	Sheet asphalt [1]	Edge	10,800	0.000230	0.000300	2,027,500	989
223	6	1:3:6	1½	do [1]	Corner	8,000	[2]	[2]		1,118
224	6	1:3:6	4	Bituminous concrete	Edge	8,000	[3]	[3]		2,086
225	6	1:3:6	4	do	Corner	8,000	[3]	[3]		1,738
226	6	1:3:6	2	do	Edge	8,000	[3]	[3]		1,141
227	6	1:3:6	2	do	Corner	8,000	[3]	[3]		901
228	4	1:3:6	2	Topeka	Edge	2,000	[4]	[4]		1,384
229	4	1:3:6	2	do	Corner	2,000	[5]	[5]		798
230	4	1:1½:3	2	do	Edge	[6]				3,365
231	4	1:1½:3	2	do	Corner	[6]				3,817
232	6	1:3:6	2	do	Edge	9,410		[7]		2,001
233	6	1:3:6	2	do	Corner	8,000	[2]	[2]		1,708
234	6	1:1½:3	2	do	Edge	11,850	.000306	.000324	4,330,000	4,268
235	6	1:1½:3	2	do	Corner	9,500	.000284	.000305	4,390,000	4,418
236	8	1:3:6	2	do	Edge	16,100	.000225	.000257	3,270,000	1,673
237	8	1:3:6	2	do	Corner	11,600	.000330	.000379	2,815,000	1,849
238	8	1:1½:3	2	do	Edge	25,250	.000242	.000267	4,685,000	5,100
239	8	1:1½:3	2	do	Corner	25,700	.000290	.000294	4,040,000	4,458
240	4	1:1½:3			Edge	[8]	.000155	.000208	4,450,000	4,350
241	4	1:1½:3			Corner	[8]	.000150		4,590,000	5,076
242	6	1:1½:3			Edge	11,650	.000272	.000291	4,200,000	4,480
243	6	1:1½:3			Corner	14,000	.000258	.000370	4,410,000	5,213
244	6	1:3:6			Edge	9,200	.000218	.000283	2,805,000	1,803
245	6	1:3:6			Corner	9,960	.000151		3,290,000	1,832
246	8	1:1½:3			Edge	27,100	.000292	.000302	4,778,000	4,857
247	8	1:1½:3			Corner	24,700	.000267	.000495	4,190,000	3,701
248	8	1:3:6			Edge	14,275	.000242	.000284	2,235,000	1,160
249	8	1:3:6			Corner	15,075	.000269	.000360	2,455,000	1,385

[1] Laid on 1½-inch binder course.
[2] Static load broke corner off.
[3] Static load cracked base.
[4] Static load caused new cracks; badly cracked in rolling.
[5] Static load caused 0.65 inch deflection; badly cracked in rolling.
[6] Too badly broken to test.
[7] Not obtained.
[8] Static load, 8,000.

TABLE III.—*Test data on rigid slabs*—Continued.

LAID ON WET SUBGRADE—Continued.

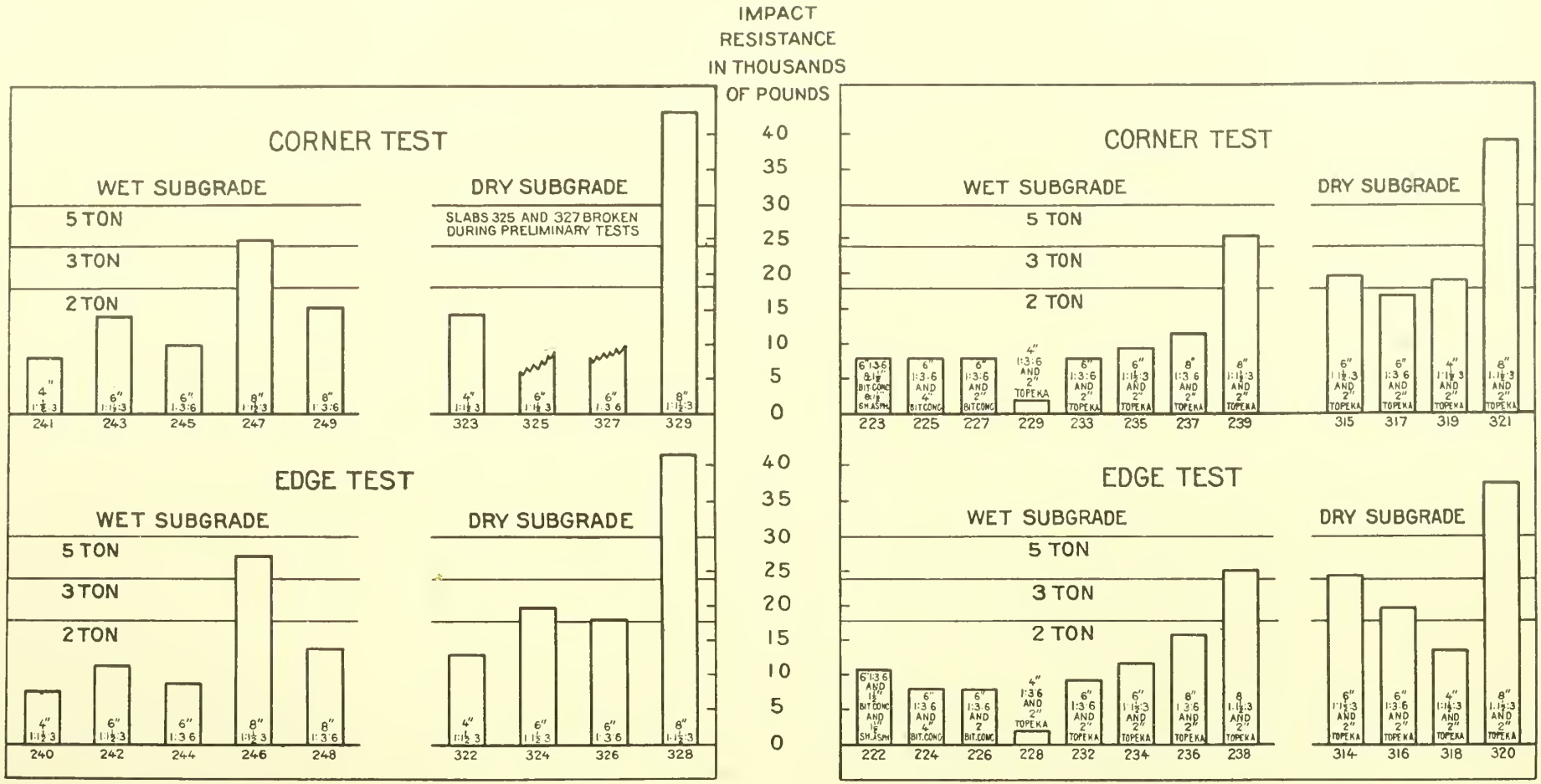
Slab No.	Construction of concrete slab. Thickness.	Mix.	Reinforcing.	Point of application of load.	Maximum impact resisted.	Maximum unit fiber deformation. Elastic.	At failure.	Average modulus of elasticity at failure.	Average crushing strength of 3 cylinders.
	Inches.				*Pounds.*			*Pounds per sq. in.*	*Pounds per sq. in.*
250	4	1:1½:3	1 layer No. 6 (0.21 per cent)	Edge	8,000	----------	0.000310	4,270,000	4,400
251	4	1:1½:3	....do	Corner	10,730	0.000265	.000307	4,125,000	4,330
252	4	1:1½:3	2 layers No. 6 (0.42 per cent)	Edge	7,550	.000203	.000365	4,250,000	4,243
253	4	1:1½:3	....do	Corner	8,500	.000260	.000275	4,270,000	4,740
254	4	1:1½:3	2 layers No. 8 (0.66 per cent)	Edge	7,790	.000293	.000360	3,970,000	4,326
255	4	1:1½:3	....do	Corner	10,260	.000286	.000309	4,125,000	4,163
256	4	1:1½:3	1 layer No. 10 (0.41 per cent)	Edge	7,025	.000278	.000301	4,422,000	4,643
257	4	1:1½:3	....do	Corner	7,790	.000253	.000315	4,610,000	4,961
258	4	1:1½:3	2 layers No. 10 (0.82 per cent)	Edge	8,300	.000283	.000343	4,120,000	3,976
259	4	1:1½:3	....do	Corner	10,950	.000281	.000317	4,650,000	4,184
260	6	1:1½:3	1 layer No. 6 (0.19 per cent)	Edge	16,600	.000267	.000298	4,255,000	4,733
261	6	1:1½:3	....do	Corner	16,200	.000284	----------	4,400,000	4,583
262	6	1:1½:3	2 layers No. 6 (0.38 per cent)	Edge	19,150	.000293	.000327	3,930,000	4,223
263	6	1:1½:3	....do	Corner	21,500	.000298	.000314	4,390,000	4,200
264	6	1:1½:3	2 layers No. 8 (0.46 per cent)	Edge	17,700	.000309	.000315	3,940,000	4,410
265	6	1:1½:3	....do	Corner	17,000	.000247	.000281	4,495,000	5,530
266	6	1:1½:3	2 layers No. 10 (0.58 per cent)	Edge	16,350	.000278	.000297	4,030,000	4,446
267	6	1:1½:3	....do	Corner	14,560	.000308	.000414	4,595,000	4,572

LAID ON DRY SUBGRADE.

Slab No.	Construction. Concrete. Thickness.	Mix.	Reinforcing	Top course. Thickness.	Material.	Point of application of load.	Maximum impact resisted.	Maximum unit fiber deformation. Elastic.	At failure.	Average modulus of elasticity at failure.	Average crushing strength of 3 cylinders.
	Inches.			*Inches.*			*Pounds.*			*Pounds per sq. in.*	*Pounds per sq. in.*
314	6	1:1½:3		2	Modified Topeka.	Edge	24,500	0.000235	0.000282	(1)	----------
315	6	1:1½:3		2	....do	Corner	19,950	.000298	.000371	(1)	----------
316	6	1:3 :6		2	....do	Edge	19,800	.000296	.000319	3,340,000	2,325
317	6	1:3 :6		2	....do	Corner	17,060	.000296	.000371	3,200,000	1,794
318	4	1:1½:3		2	....do	....do	13,650	.000272	.000340	3,300,000	3,212
319	4	1:1½:3		2	....do	Edge	19,400	.000314	.000372	3,300,000	3,660
320	8	1:1½:3		2	....do	....do	37,800	.000295	.000315	3,660,000	3,940
321	8	1:1½:3		2	....do	Corner	39,800	.000292	.000312	4,640,000	3,340
322	4	1:1½:3				Edge	13,300	(2)	.000353	4,750,000	4,488
323	4	1:1½:3				Corner	14,000	.000308	.000319	4,000,000	3,938
324	6	1:1½:3				Edge	20,000	.000271	.000297	4,330,000	4,698
325	(3)										5,446
326	6	1:3 :6				Edge	18,300	.000237	.000292	2,620,000	1,632
327	(3)										1,331
328	8	1:1½:3				Edge	41,500	.000337	.000395	3,620,000	3,334
329	8	1:1½:3				Corner	42,580	.000264	.000451	4,450,000	4,993
330	4	1:1½:3	1 layer No. 6 (0.21 per cent)			Edge	13,410	.000278	.000295	4,170,000	3,493
331	4	1:1½:3	....do			Corner	11,600	.000280	.000312	4,500,000	4,709
332	4	1:1½:3	1 layer No. 10 (0.41 per cent)			Edge	11,250	.000243	.000332	4,850,000	4,183
333	4	1:1½:3	....do			Corner	11,150	.000215	.000301	4,420,000	3,752
334	6	1:1½:3	2 layers No. 6 (0.38 per cent)			Edge	20,000	.000262	.000279	4,700,000	4,769
335	6	1:1½:3	....do			Corner	23,700	.000256	(4)	4,710,000	4,806
336	(3)										4,626
337	6	1:1½:3	2 layers No. 8 (0.46 per cent)			Corner	21,800	.000244	.000277	4,330,000	3,861

Slab No.	Reinforcing of 6-inch 1:1½:3 concrete slabs.	Point of application of load.	Maximum impact resisted.	Maximum unit fiber deformation. Elastic.	At failure.	Average modulus of elasticity at failure.	Average crushing strength of 3 cylinders.
			Pounds.			*Pounds per sq. in.*	*Pounds per sq. in.*
1-R	1 layer ⅜-inch rods 3¾ inches center to center	Edge	23,050	0.000271	0.000308	4,550,000	6,068
2-R	....do	Corner	24,100	.000281	.000302	4,960,000	6,193
3-R	2 layers ⅜-inch rods 3¾ inches center to center	Edge	21,450	.000296	.000355	4,395,000	5,920
4-R	....do	Corner	29,200	.000281	.000301	4,625,000	6,508
5-R	2 layers ⅜-inch rods 7⅛ inches center to center	Edge	22,000	.000286	.000348	4,765,000	5,548
6-R	....do	Corner	24,700	.000247	----------	4,950,000	5,857
7-R	1 layer ⅝-inch rods 10¼ inches center to center	Edge	22,930	.000228	.000308	4,510,000	6,427
8-R	....do	Corner	24,900	.000273	----------	5,000,000	6,635
9-R	2 layers ⅝-inch rods 10¼ inches center to center	Edge	23,600	.000267	.000304	4,630,000	5,701
10-R	....do	Corner	28,650	.000277	.000308	4,835,000	5,833
11-R	2 layers ⅝-inch rods 20½ inches center to center	Edge	19,350	.000243	.000300	4,540,000	6,161
12-R	....do	Corner	24,650	.000287	----------	4,740,000	6,174
13-R	1 layer ⅞-inch rods 20 inches center to center	Edge	20,900	.000281	.000367	4,940,000	5,738
14-R	....do	Corner	24,150	.000297	.000399	4,155,000	5,426
15-R	2 layers ⅞-inch rods 20 inches center to center	Edge	23,850	.000247	.000348	5,125,000	5,866
16-R	....do	Corner	26,550	.000243	.000345	4,805,000	6,113
17-R	2 layers ⅞-inch rods 40 inches center to center	Edge	23,050	.000301	.000348	4,515,000	6,040
18-R	....do	Corner	27,100	.000292	.000304	4,420,000	4,880

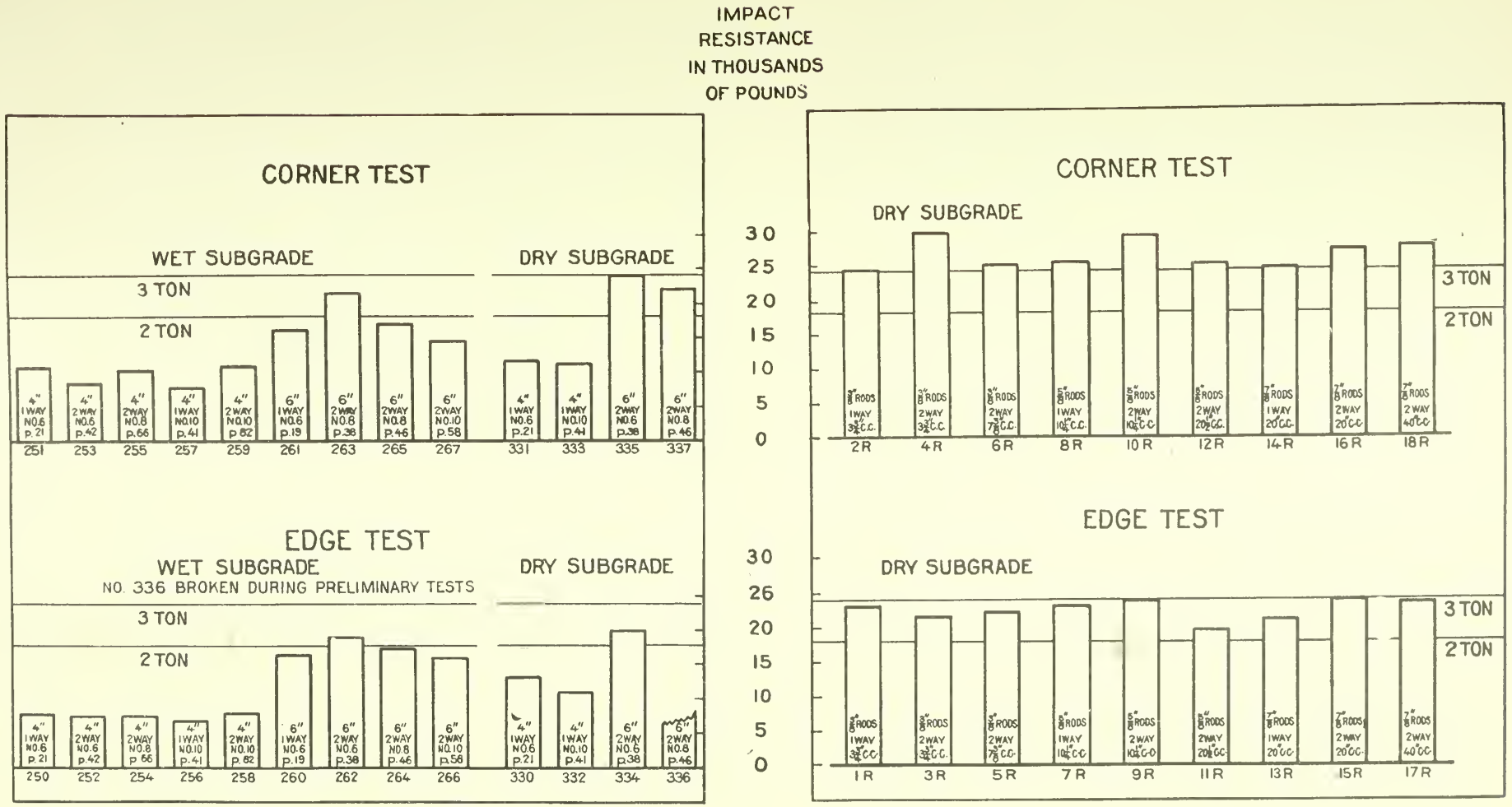
[1] No test cylinders. [2] Not obtained. [3] Broken during preliminary test last year. [4] Beyond gauge length.



PLAIN CONCRETE SLABS

CONCRETE SLABS WITH BITUMINOUS TOPS

Impact resistance of plain concrete slabs and concrete slabs with bituminous tops.



MESH-REINFORCED CONCRETE SLABS

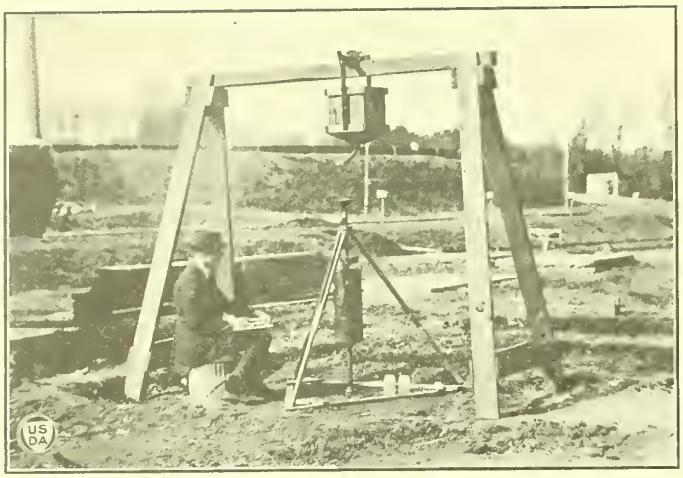
ALL SLABS OF 1:1½:3 MIX

CROSS MEMBERS NOT WELDED TOGETHER

ROD-REINFORCED CONCRETE SLABS

ALL SLABS 6 INCHES THICK 1:1½:3 MIX

Impact resistance of mesh-reinforced and rod-reinforced concrete slabs



Subgrade bearing value determination. Shot from an overhead container falls at a constant rate into a drum supported by a small bearing block. The penetration of this block into the subgrade is measured by an Ames dial.

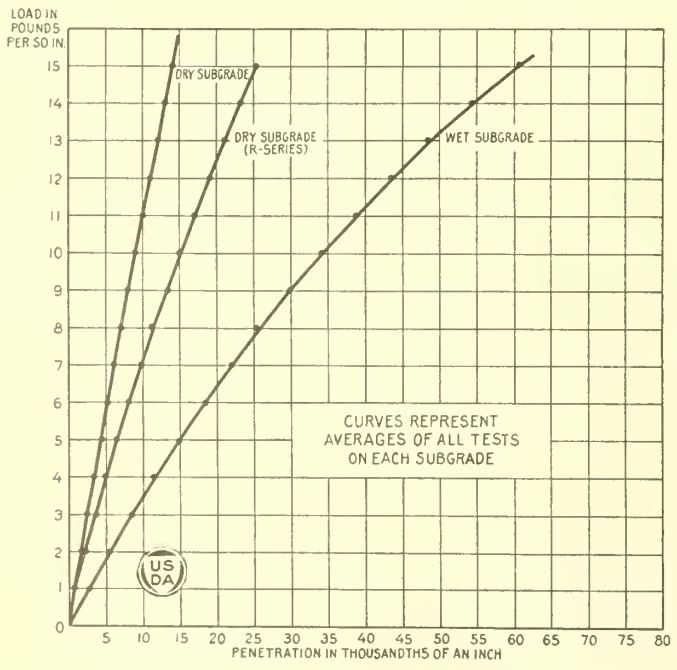
as that on the bottom of the slab in the edge test. The area broken off in the corner tests seems to depend on the condition of the subgrade, as it is noticeable that smaller areas are broken off where the subgrade is firm. It was found possible to break off a corner by an impact delivered at the quarter point of one edge; i. e., at a point 21 inches from the corner along one edge. This would seem to indicate that some corner breaks may start from impacts delivered along the edge which produce tension in the bottom of slab.

THE EFFECT OF REINFORCING.

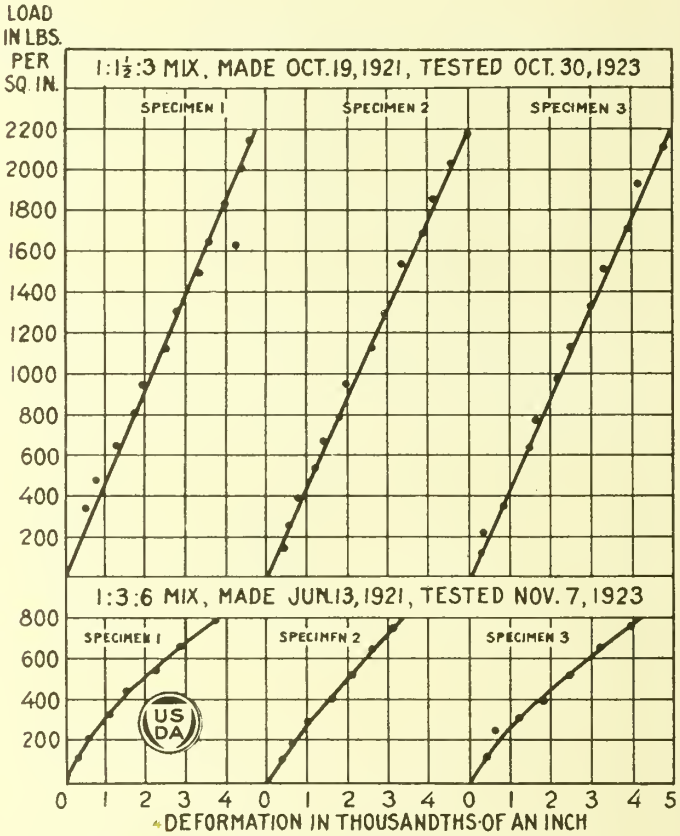
A study of the data must inevitably lead to the conclusion that fabric reinforcing of the type used in these tests at or near the center of a road slab does not appreciably increase its resistance to impact. Observation at the time of test bears this out. In the percentages used in these test sections, no great additional strength was noticed after the formation of the first crack, and in no case did repeated applications of the same impact which caused failure fail to open up a crack into which water and mud could work and attack the steel.

Results of the tests on slabs reinforced with deformed bars show a definite increase in strength, particularly in the corner tests where the steel was in a better position to take tensile stress. The compression cylinders indicate a somewhat stronger concrete in this group of slabs for no known reason, the same care having been taken in placing all of the slabs. To offset this, however, the subgrade shows a lower bearing value than the other sections on the dry subgrade.

In the corner test the slabs having 1 per cent of steel show about 13 per cent more strength than the slabs having 0.5 per cent. A study of the behavior of the breaks after failure leads to the conclusion that, for a given percentage of steel, small rods closely spaced are more effective than large rods widely spaced.


LOAD IN POUNDS PER SQ IN.
DRY SUBGRADE
DRY SUBGRADE (R-SERIES)
WET SUBGRADE
CURVES REPRESENT AVERAGES OF ALL TESTS ON EACH SUBGRADE
PENETRATION IN THOUSANDTHS OF AN INCH

Load-penetration curves. Soil bearing value tests.


LOAD IN LBS. PER SQ. IN.
1:1½:3 MIX, MADE OCT. 19, 1921, TESTED OCT. 30, 1923
SPECIMEN 1
SPECIMEN 2
SPECIMEN 3
1:3:6 MIX, MADE JUN. 13, 1921, TESTED NOV. 7, 1923
DEFORMATION IN THOUSANDTHS OF AN INCH

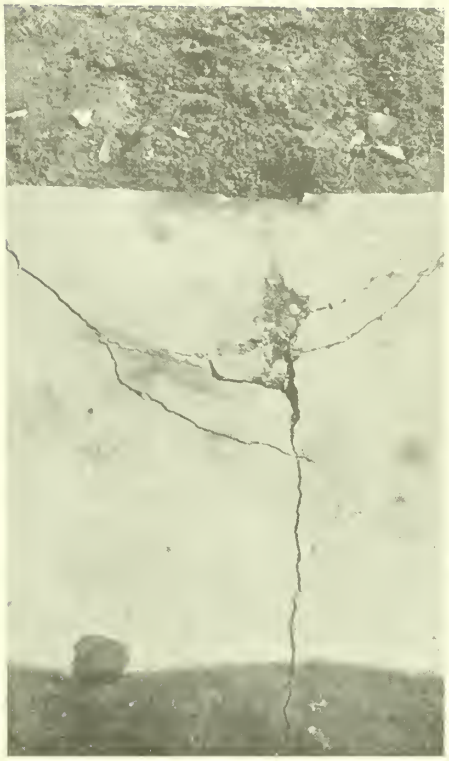
Typical modulus of elasticity curves obtained from 6 by 12 inch compression cylinders.

In the edge test there was little additional resistance to the formation of the first crack. This was to be expected in view of the fact that the steel was placed in the upper part of the slab. But a very marked resistance to further failure was apparent in all the slabs. In no case was it possible to develop the crack through to the top of the slab, although a complete transverse crack showed at the bottom. Additional very high impacts failed to break down this resistance. This was not true of the mesh-reinforced slabs.

LITTLE EVIDENCE OF CUSHIONING BY BITUMINOUS TOPS.

In examining the data with the idea of finding out to what extent bituminous tops on concrete cushion the impact of traffic, there are a number of facts to be borne in mind. Everyone is familiar with the springi-

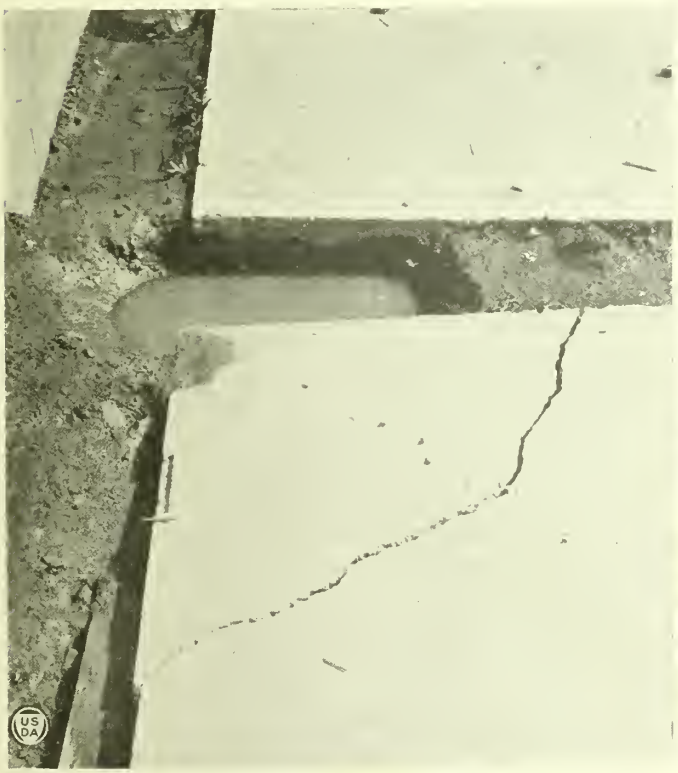
Transverse crack, the result of impact on a 6-inch 1:1½:3 plain concrete slab.



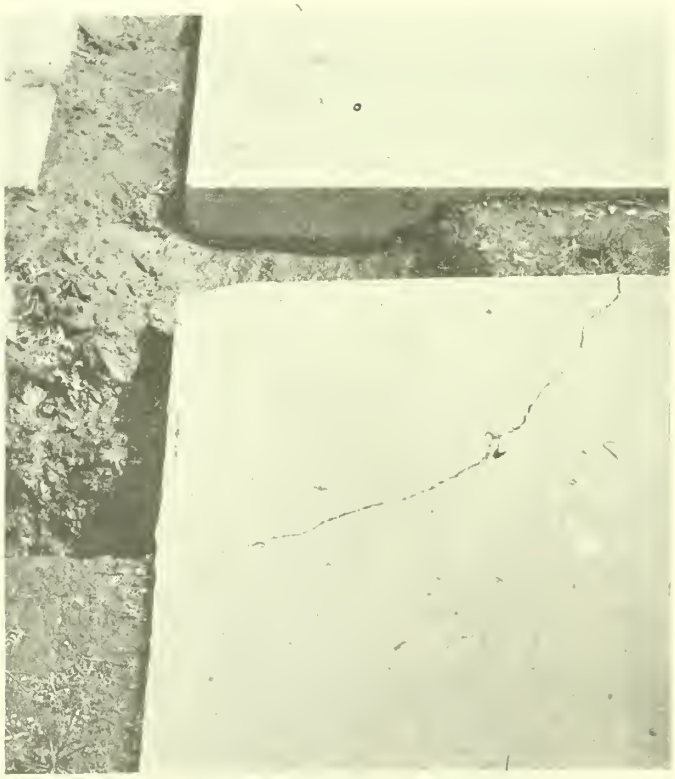
Transverse crack with corners broken off in a 4-inch, mesh-reinforced concrete slab.

Transverse and longitudinal crack resulting from impact on an 8-inch, 1:3:6 plain concrete slab.

Typical failures in edge tests of concrete slabs laid on wet subgrade.

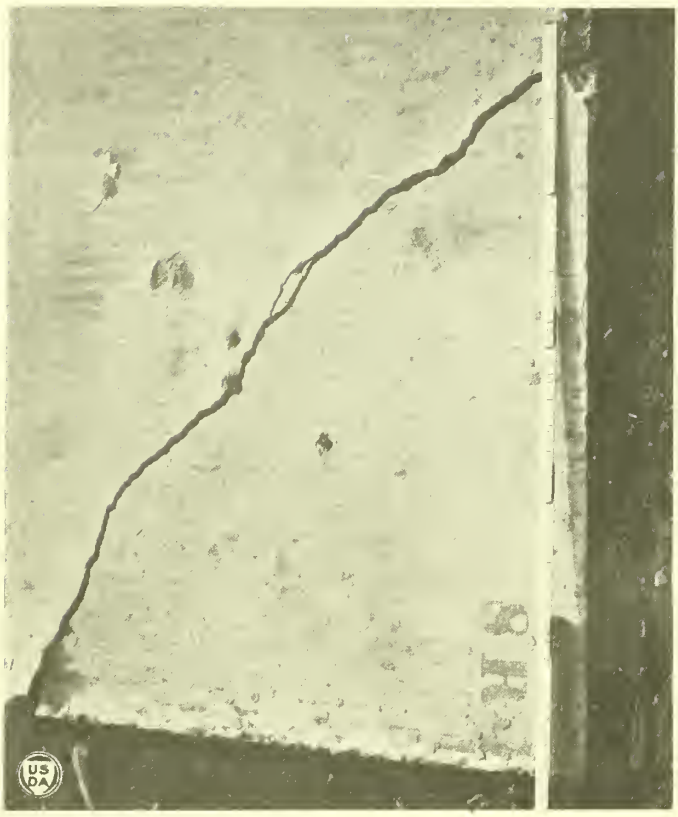


6-inch mesh-reinforced concrete slab.

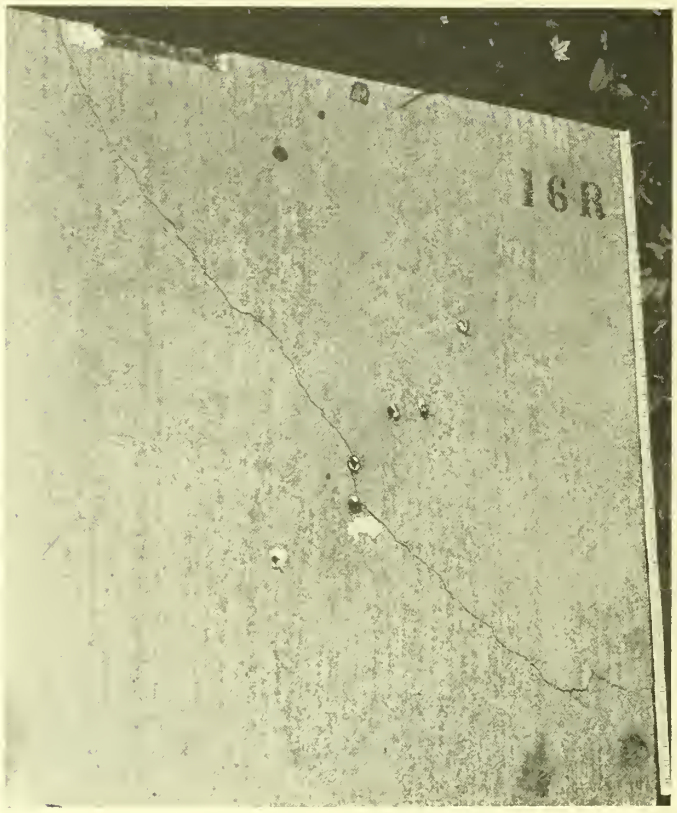


6-inch plain concrete slab.

Typical failures in corner tests of concrete slabs laid on wet subgrade.

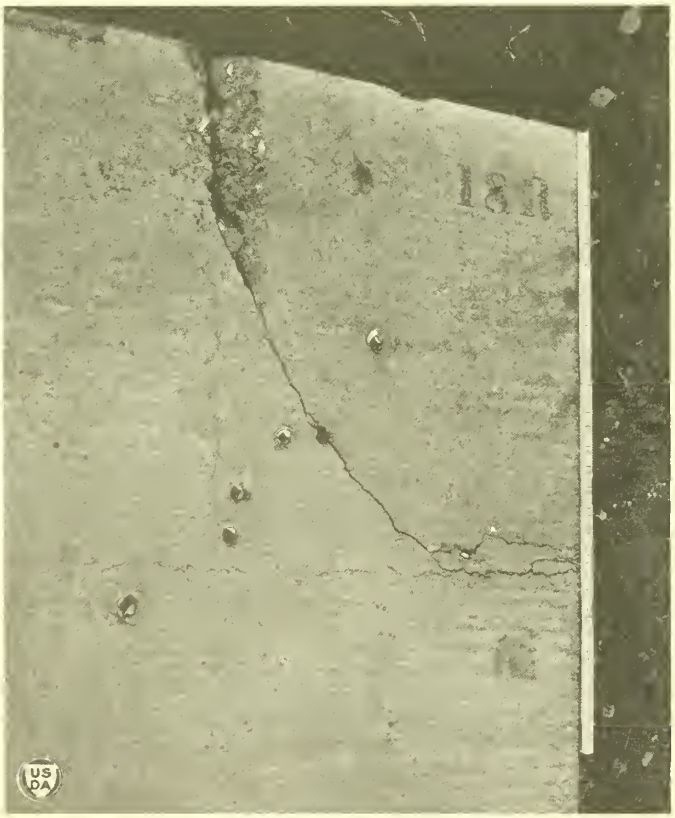


Low percentage of steel and one-way placing result in wide crack under additional impact.

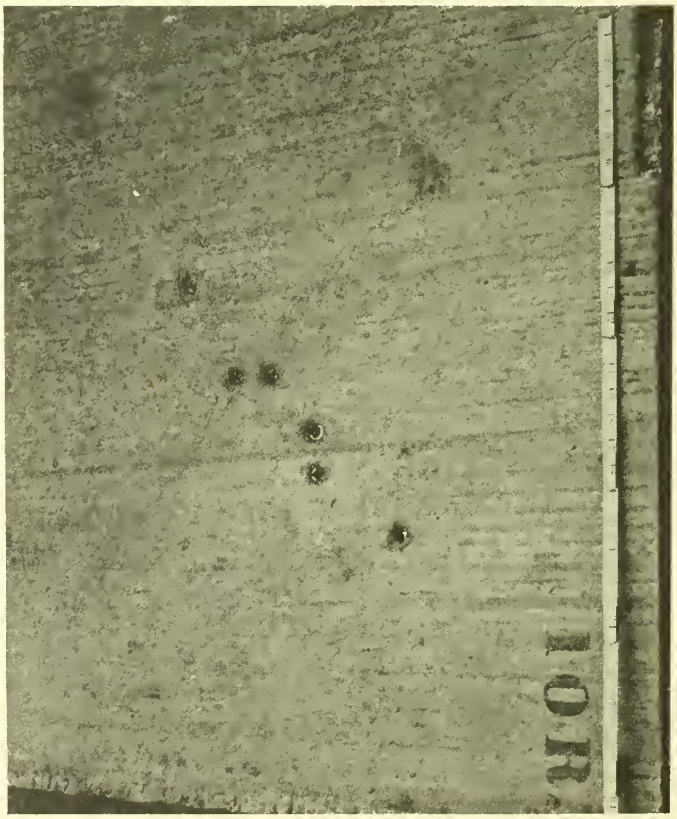


Effect of a larger percentage of steel—a small crack which does not open up under additional impact.

Typical failures of rod-reinforced sections in corner test.

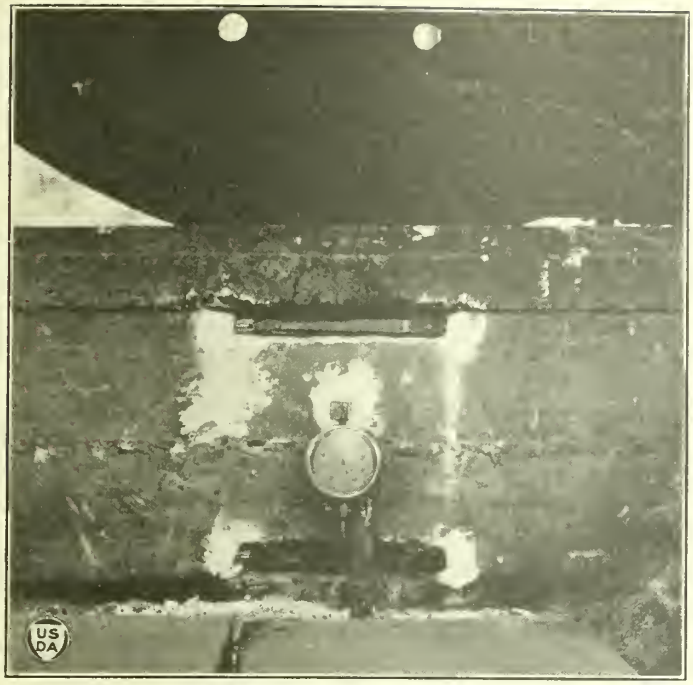


Effect of large rods apparent in this crack.



High percentage of steel and two-way placing result in second crack about 7 inches nearer the corner than the first. First crack refused to open under additional blows.

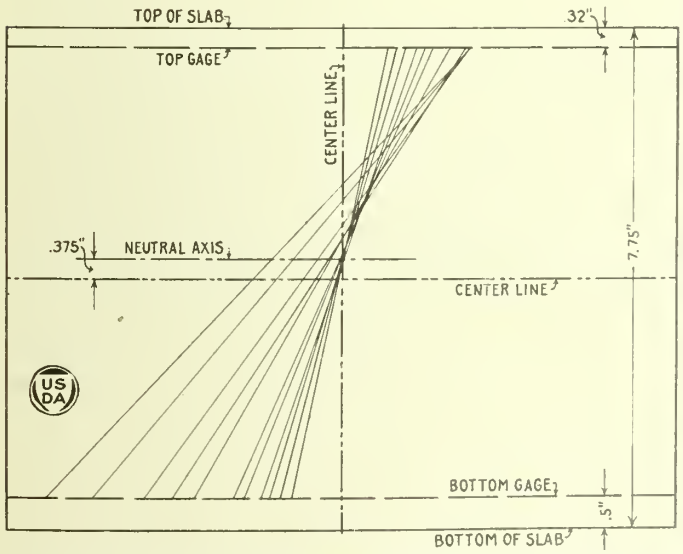
Typical failures of rod-reinforced sections in corner test.



Set-up of strain gauges and deflection dial for the edge test. Topeka top beginning to crack.

ness of an asphalt city pavement on a hot day, and it is easy to jump to the conclusion that such a surface acts as a cushion to the blows of traffic.

But even in this springy state, while undoubtedly the bituminous mixture yields readily to a steady pressure, it is doubtful if it moves appreciably under an impact whose duration is but a small fraction of a second. To cushion a wheel impact, something must act to bring the wheel slowly to rest; in this case the asphalt top must move and move considerably. Observations during the impact tests show that it does move, but under a single blow its movement is very slight indeed. The results of all comparable slabs with and without asphalt tops (2-inch modified Topeka) show little evidence of cushioning due to these tops.



Determination of neutral axis from strain gauge measurement on concrete slabs with bituminous tops. The neutral axis remains fixed until the lower fibers begin to fail.

The temperature is an extremely important factor in the behavior of bituminous mixtures. The tests herein described were made during the summer months and the temperatures of the asphalt varied from 25° to 32° C. (77° to 90° F.) Undoubtedly at higher temperatures more movement would take place in the material under a wheel impact.

Table IV shows the results from which these conclusions were drawn.

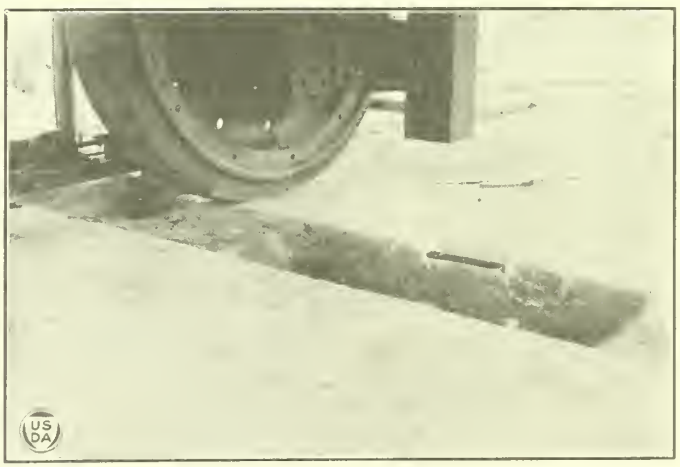
TABLE IV.—*Data on cushioning of bituminous tops.*

Concrete.		Plain.		With 2-inch Topeka top.	
Mix.	Thickness.	Height of drop.	Impact of 2-ton truck wheel.	Height of drop.	Impact of 2-ton truck wheel.
Dry subgrade:	*Inches.*	*Inches.*	*Pounds.*	*Inches.*	*Pounds.*
1 : 1½ : 3	4	0.97	11,440	0.95	12,060
1 : 3 : 6	6	.54	18,300	.62	19,800
1 : 1½ : 3	6	.68	20,000	.60	24,500
1 : 1½ : 3	8	1.15	41,500	1.23	37,800
Wet subgrade:					
1 : 1½ : 3	4	(1)	(1)		
1 : 3 : 6	6	.48	9,200	.47	9,460
1: 1½ : 3	6	1.06	11,600	1.18	11,330
1 : 3 : 6	8	.40	8,660	.46	8,975
1 : 1½ : 3	8	1.53	27,100	1.92	25,250

[1] Failed under static load.

To draw an absolute conclusion regarding cushioning, every condition of subgrade, sprung weight, unsprung weight, height of drop, etc., should be identical. With these conditions constant, a top which acts as a cushion should show a lower impact force for a given height of drop. The slabs listed in the table are compared in this manner, and with one exception they show no evidence of this effect. The 8-inch slabs of the 1 : 1½ : 3 mix show indications of such an action toward the end of the test where very heavy impacts were being applied. At the impact which broke the 4-inch and 6-inch slabs no cushioning is apparent.

It is realized that perfect conditions for comparison did not obtain in the case of these tests, especially in the matter of subgrade support. They more nearly duplicate road conditions, however, and for this reason should be more valuable than some laboratory tests which do not in any way compare with truck-wheel impact.



Set-up for the corner test. Strain gauges on top of slab are for determination of maximum tensile stress. Those in edge of slab locate the neutral plane.

EFFECT OF BITUMINOUS TOPS ON SLAB STRENGTH.

The resistance of the concrete slabs of the different mixes and thickness is not, in general, increased by the addition of a bituminous top of the thickness used in these tests, except possibly in the case of the 4-inch and 6-inch bases on the dry subgrade. Several showed distinctly less resistance, but it should be noted that these were slabs laid on the wet subgrade and in several cases there was evidence of deterioration in the concrete under the bituminous layer. Evaporation of moisture from the concrete was prevented and this may have caused the weakened condition of these bases. The 1 : 3 : 6 mix seemed to be affected to a greater degree than the 1 : 1½ : 3 mix.

There was very little bond between the concrete bases and the bituminous surfaces of any of the types used. Certainly there was not enough to resist appreciable horizontal shearing stresses. These observations were borne out by the strain gage readings in the edge test, which showed no raising of the neutral plane of the slab when a bituminous top was added. The average of all of the edge tests on these slabs on the dry subgrade showed the neutral plane to be 0.37 inch above the horizontal center plane of the slab. The same data for the concrete slabs without bituminous tops showed an average measurement of 0.39 inch above the horizontal center plane. The fact that all the slabs showed this plane to be above center may be due to a lesser density of the concrete next to the subgrade.

Topeka tops proved to be extremely tough and resistant to cracking, more so than the sheet asphalt with binder course. Even after the failure of the concrete bases, the bituminous layer bridged the crack and withstood the impact, materially reducing the destructive effect of the succeeding blows. On the dry subgrade, where the foundation support was good, these Topeka tops withstood very severe punishment with little deformation or cracking.

MOTOR VEHICLE REGISTRATION 15,092,177.

$225,784,932 AMOUNT OF ANNUAL LICENSE FEES AND GASOLINE TAXES.

By ANDREW P. ANDERSON, Highway Engineer, U. S. Bureau of Public Roads.

THE YEAR 1923 has shown a truly remarkable increase in the number of motor vehicles. A total of 15,092,177 passenger cars, motor trucks, and commercial vehicles were registered in the 48 States and District of Columbia during the registration year 1923. This is an actual increase over 1922 of 2,853,802 registrations, 408,136 more cars than the total number registered in 1915, or 1,078,722 more than the greatest previous increase recorded during one year.

There are now approximately 2 motor vehicles for every 15 persons of our total population, and if the increase during the present year should be as great as during the past, we shall have at the end of the year 1 motor vehicle for every 6 persons of our total estimated population—enough to transport our entire population at one time should the necessity arise.

The statistics of motor-vehicle registrations read more like a tale from Baron Munchausen or the Arabian Nights than prosaic records of actual facts. In 1913, the earliest year for which fairly reliable data are available, the total number of vehicles registered was only 1,258,062, or approximately one-twelfth of the present number. In 1919 the registrations had increased to 7,565,446, or almost exactly one-half of the total for 1923. This rate of increase obviously can not continue indefinitely, but the records of the past give no clue as to when or how it will cease.

Owners of these motor vehicles are insistent in their demand for improved roads, and they are apparently contributing substantially to the funds available for road work, especially for the maintenance of State highways. Total gross receipts from registration fees, together with the licenses of drivers, chauffeurs, etc., which in 1913 amounted to only $8,192,253, in 1923 amounted to $188,970,992.24. Thus, while the number of motor vehicle registrations increased only 12 times during the 10-year period, the funds so collected increased 23 times.

Practically the whole amount of the license fees collected, after deducting the sums required to pay the cost of registration, is now devoted to road work or to the financing of debts incurred for road construction, and by far the greater part of these funds is expended by or under the supervision of the State highway departments. Of the total of $188,970,992.24 collected during 1923, $153,226,636.16 has been or will be applied to road work conducted by or under the supervision of the State highway departments, although in a few States this supervision or control is not as complete as it should be.

AVERAGE MOTOR TRUCK FEE MORE THAN TWICE THAT OF AVERAGE PASSENGER CAR.

Returns from 29 States in which the records permit the segregation of fees paid for motor trucks from those paid for passenger cars show that the fee paid for the average truck is more than twice as great as the average fee paid per passenger car. In these 29 States a total of $24,020,784.89 was paid in fees for 1,103,076 motor trucks and only $91,031,927.06 for 8,671,635 passenger cars.

In addition to the direct fees paid by the motor-vehicle owner or operator, he also pays a large amount in indirect fees or taxes. In most States motor vehicles are taxed as personal property. Many cities levy wheel taxes or additional registration fees. The Federal Government collects an excise tax of 5 per cent of the manufacturer's sale price on all passenger cars, parts, tires, and accessories and 3 per cent on the sale price of motor trucks. And over and above all these taxes the majority of the States now tax gasoline or motor fuel. This form of tax has proved to be most popular with the State legislatures. First adopted by the States of Oregon, New Mexico, and Colorado in 1919, this form of taxation is now in force in 35 States, the tax ranging from 1 to 4 cents

TABLE I.—*Motor vehicle registration, licenses, and revenues, registration year 1923.*

State.	Grand total motor cars.	Private passenger cars.	Motor trucks.	Taxis, busses, and cars for hire.[1]	Motor cycles.	Registration fees, licenses, and permits.		Amount of registration fees paid.		Total registration, 1922.	Per cent increase during 1923.
						Total gross receipts.	Amount applicable to highway work by or under supervision of State highway department.	Private passenger cars.	Motor trucks.		
Alabama	126,642	109,535	13,845	3,262	599	$1,541,017.56	$1,204,449.02			90,052	40.4
Arizona	49,175	42,176	6,565	434	392	281,670.75	281,670.75			38,034	29.3
Arkansas	113,300	102,000	11,300		300	1,435,090.00	430,527.12	$1,224,000.00	$192,100.00	84,596	33.9
California	1,100,283	1,056,756	43,527		14,694	10,608,544.00	4,906,015.00	9,081,836.00	814,138.00	861,807	27.7
Colorado	188,956	175,669	13,287	(2)	2,473	1,126,218.55	534,953.81	898,666.40	153,741.57	162,328	16.4
Connecticut	181,748	148,791	29,140	3,817	4,450	4,329,432.16	4,329,432.16	2,302,154.23	956,368.93	152,977	18.8
Delaware	29,977	24,709	5,268		467	516,209.00	516,209.00	287,950.00	108,379.00	24,560	22.1
District of Columbia[3]	74,811	[3] 65,681	[3] 7,187	[3] 1,943	1,772	357,918.00				52,792	41.7
Florida	151,990	125,140	23,530	3,320	975	1,963,065.99	1,394,528.58			116,170	30.8
Georgia	173,889	151,325	22,469	95	1,011	2,156,406.08	2,095,762.60	1,756,219.20	344,831.04	143,423	21.2
Idaho	62,379	57,200	5,179		655	914,014.58	229,840.14	807,678.15	87,309.43	53,874	15.8
Illinois	969,331	847,005	122,326		7,611	9,653,796.04	9,653,796.04	7,132,472.16	1,820,379.48	781,974	23.9
Indiana	583,342	510,114	73,228		6,042	3,693,715.00	3,492,498.00	2,651,084.00	794,003.00	469,939	24.1
Iowa	571,061	534,796	36,265		3,044	8,827,062.99	[4] 8,000,000.00			500,158	14.2
Kansas	375,594	349,038	26,556	(5)	1,950	3,435,606.00	[6] 1,750,000.00			327,194	14.8
Kentucky	198,377	177,834	20,543		844	2,678,732.89	2,678,732.89			154,021	28.8
Louisiana	136,622	116,003	20,619		400	2,191,240.81	2,191,240.81	1,800,186.81	336,000.00	102,284	33.6
Maine	108,609	90,177	15,614	2,818	1,400	1,660,268.17	1,474,383.39			92,539	17.4
Maryland	169,351	153,661	11,609	4,081	4,846	3,536,955.20	3,183,259.68	2,230,333.05	461,539.95	165,624	[18] 24.0
Massachusetts	481,150	407,645	73,505		11,033	6,989,633.25	6,639,155.42	4,314,529.50	1,117,834.00	385,231	24.8
Michigan	730,658	657,148	72,000	1,510	4,165	10,500,786.05	4,741,624.91	8,135,757.89	[7] 1,225,958.00	578,210	26.4
Minnesota	448,187	399,404	48,783		3,220	7,316,772.03	7,316,772.03	6,212,601.93	959,495.66	380,557	17.8
Mississippi	104,286	93,846	10,440	(2)	114	1,077,616.22	580,852.12			77,571	34.4
Missouri	476,598	430,340	46,258		2,570	4,016,383.60	[8] 4,016,383.60			392,523	21.5
Montana	73,828	65,449	8,379		374	729,621.50	[9] 73,325.64	604,663.75	93,162.75	62,650	17.8
Nebraska	286,053	259,382	26,671		1,608	3,353,175.32	2,932,242.63	2,754,430.61	498,750.99	256,654	11.4
Nevada	15,699	13,699	2,000		112	153,888.10	[10] 144,992.15	119,798.10	30,000.00	12,116	29.6
New Hampshire	59,604	52,608	6,996	(5)	1,987	1,571,326.96	1,464,096.88			48,406	23.2
New Jersey	430,958	330,552	89,105	11,301	8,811	7,653,780.37	7,515,116.03	3,069,466.75	2,407,423.50	342,286	25.9
New Mexico	32,032	29,032	3,000		215	295,000.00	280,250.00	251,995.00	36,000.00	25,473	25.7
New York	1,204,213	962,681	203,846	37,686	22,153	19,862,441.52	14,896,831.14	11,689,802.97	5,391,418.25	1,002,293	20.1
North Carolina[11]	246,812	225,488	21,324	(2)	1,300	3,728,044.72	[4] 3,700,000.00			182,550	35.2
North Dakota	109,266	105,958	3,287	21	645	760,852.45	760,444.45			99,052	10.3
Ohio	1,069,100	927,200	141,900		15,000	9,662,370.29	4,832,962.25			858,716	24.5
Oklahoma	307,000	288,424	18,576	(2)	823	3,217,770.84	[12] 2,895,000.00			249,659	23.0
Oregon	165,962	[13] 152,135	12,987	840	3,140	4,069,609.40	[14] 2,924,707.05			134,125	23.7
Pennsylvania	1,043,770	[15] 969,361	74,409		19,220	15,844,303.80	15,844,303.80	9,944,691.80	3,410,031.50	829,737	25.8
Rhode Island	76,312	60,620	13,930	1,762	1,575	1,286,659.47	1,196,909.47	727,704.72	275,240.27	66,083	15.5
South Carolina	127,467	115,892	11,575		547	902,608.69	722,086.95	728,644.01	153,593.69	95,239	33.8
South Dakota	131,700	121,164	10,536		471	1,130,959.27	1,055,175.80			125,241	5.2
Tennessee	173,365	154,181	19,184		751	2,049,653.27	2,028,806.14			135,716	27.8
Texas	688,233	[16] 688,233	(2)		3,346	5,441,508.59	2,368,569.43			526,238	30.8
Utah	59,525	51,625	7,900		766	430,104.72	[14] 430,104.72			49,164	21.1
Vermont	52,776	49,420	3,356		839	938,860.30	860,803.03	674,868.63	83,394.50	43,881	20.2
Virginia	218,896	187,977	30,919		1,813	3,200,161.66	3,200,161.66	2,422,993.72	482,670.25	168,000	30.3
Washington	258,264	218,580	37,100	2,584	3,560	3,898,597.77	3,741,167.81	2,726,121.45	843,041.92	210,716	22.6
West Virginia	157,924	143,548	[17] 7,456	6,920	1,353	2,608,508.37	[8] 2,608,508.37	1,940,093.23	229,157.66	112,763	40.0
Wisconsin	457,271	422,718	34,553	(2)	5,645	4,958,933.55	4,693,887.30	4,227,180.00	625,619.55	382,542	19.5
Wyoming	39,831	35,294	4,537	(2)	291	414,096.39	[14] 414,096.39	314,003.00	89,202.00	30,637	30.1
Total	15,092,177	13,457,214	1,552,569	82,394	171,372	188,970,992.24	153,226,636.16	91,031,927.06	24,020,784.89	12,238,375	23.6

[1] Where no data are given these vehicles are not registered as a separate class but included with passenger cars or trucks.
[2] Included with passenger cars.
[3] Includes re-registrations, but does not include nonresident registrations.
[4] Approximate.
[5] Included with trucks.
[6] Approximate amount available for State-aid road work.
[7] Includes receipts from taxicabs, motor busses, and cars for hire.
[8] For State highway work and financing State highway bonds.
[9] State's share for period Jan. 1 to Apr. 1 when new law becomes effective.
[10] Includes $48,115 used to finance State highway bonds.
[11] All data for the State of North Carolina are for the first 6 months of the registration year, which begins on July 1.
[12] To be expended by counties under general regulation made by State highway department.
[13] Includes ambulances and commercial cars under 1-ton capacity.
[14] To finance State highway bonds.
[15] Includes 88,650 commercial vehicles having a chassis weight of less than 2,000 pounds.
[16] Includes motor trucks.
[17] Solid-tire vehicles only.
[18] Nonresident registrations included in both years for this computation.

per gallon. In Massachusetts legislation providing for a tax of 2 cents per gallon has been passed, but is inoperative pending a referendum at the general election next November. In Minnesota provision has been made to submit to the voters at the next general election a constitutional amendment permitting such a tax to be levied.

The total gross receipts from gasoline taxes amounted to $36,813,939.61 during the calendar year 1923. Of this total, $21,528,559.18 is applicable to highway work conducted by or under the direct supervision of the State highway departments. Only one State, North Dakota, devotes no part of the revenue from this tax to highway work, and only two States, Alabama and Pennsylvania, place no part of these revenues under the direct control of the State highway department.

In compiling the statistics the registration year has been used as the basis rather than the calendar year. In most States the registration year coincides with the calendar year and in only one State, North Carolina, does the registration year differ very widely from the calendar year. In all cases where the registration year ends later than January 31 the registration data given are for the period from the beginning of the registration year to the close of the calendar year 1923. It is believed that this method serves to give the most reliable information as to the actual number of cars in use.

Tables I to IV, inclusive, show in more detail the statistics of motor-vehicle registration, revenues, and gasoline taxes for the year 1923 and also comparative data as to registrations and gross receipts for each of the years 1913 to 1923, inclusive.

TABLE II.—*Summary of combined passenger car and motor truck registrations for years 1913 to 1923, inclusive.*

State.	1913	1914	1915	1916	1917	1918	1919	1920	1921	1922	1923
Alabama	5,300	8,672	11,634	21,636	32,873	46,171	58,898	74,637	82,366	90,052	126,642
Arizona	3,613	5,040	7,753	12,300	19,890	23,905	28,979	34,601	35,611	38,034	49,175
Arkansas	3,583	5,642	8,021	15,000	28,693	41,458	49,450	59,082	67,408	84,596	113,300
California	[1] 100,000	123,504	163,797	232,440	306,916	407,761	477,450	583,623	680,614	861,807	1,100,283
Colorado	13,000	17,756	28,894	43,296	87,460	83,244	104,865	129,255	145,739	162,328	188,956
Connecticut	23,200	27,786	41,121	56,048	74,645	86,067	102,410	119,134	134,141	152,977	181,748
Delaware	2,440	3,050	5,052	7,102	10,700	12,955	16,152	18,300	21,413	24,560	29,977
District of Columbia	4,000	4,833	8,009	13,118	15,493	30,490	35,400	34,161	40,625	52,792	[2] 74,811
Florida	[3] 3,000	[3] 3,368	[3] 10,850	20,718	[3] 27,000	54,186	55,400	73,914	97,957	116,170	151,990
Georgia	[3] 20,000	20,915	25,000	46,025	70,324	104,676	137,000	146,000	131,976	143,423	173,889
Idaho	2,113	3,346	7,071	12,999	24,731	32,289	42,220	50,861	51,294	53,874	62,379
Illinois	94,656	131,140	180,832	248,429	340,292	389,620	478,438	568,924	663,348	781,974	969,331
Indiana	45,000	66,500	96,915	139,065	192,194	227,160	227,255	333,067	400,342	469,939	583,342
Iowa	70,299	106,087	145,109	198,587	254,462	278,313	364,043	437,378	461,084	500,158	571,061
Kansas	34,550	49,374	72,520	112,122	159,343	189,163	228,600	294,159	289,539	327,194	375,594
Kentucky	7,210	11,766	19,500	31,500	47,420	65,884	90,008	112,683	126,802	154,021	198,377
Louisiana	[1] 10,000	[1] 12,000	11,380	17,000	28,394	40,000	51,000	73,000	77,885	102,284	136,622
Maine	11,022	15,700	21,545	30,972	41,499	44,572	53,425	62,907	77,527	92,539	108,609
Maryland	14,217	20,213	31,047	44,245	60,943	74,666	95,634	102,841	136,249	165,624	169,351
Massachusetts	62,660	77,246	102,633	136,809	174,274	193,497	247,182	274,498	360,732	385,231	481,150
Michigan	54,366	76,389	114,845	160,052	247,006	262,125	325,813	412,717	476,452	578,210	730,658
Minnesota	46,000	67,862	93,269	[4] 46,000	[5] 54,000	204,458	259,741	324,166	323,475	380,557	448,187
Mississippi	3,850	5,694	9,669	25,000	36,600	48,400	59,000	68,486	65,039	77,571	104,286
Missouri	38,140	54,468	76,462	103,587	147,528	188,040	244,363	297,008	346,437	392,523	476,598
Montana	5,916	10,200	14,540	25,105	42,749	51,053	59,324	60,650	58,785	62,650	73,828
Nebraska	13,411	16,385	59,000	101,200	148,101	173,374	200,000	219,000	238,704	256,654	286,053
Nevada	1,091	1,487	2,009	4,919	7,160	8,159	9,305	10,464	10,821	12,116	15,699
New Hampshire	8,237	9,571	13,449	17,508	22,267	24,817	31,625	34,680	42,039	48,406	59,604
New Jersey	51,360	62,961	81,848	109,414	141,918	155,519	190,873	227,737	272,994	342,286	430,958
New Mexico	1,898	3,090	5,100	8,228	14,086	17,647	18,082	22,100	22,559	25,473	32,032
New York	134,495	168,223	255,242	314,222	406,016	459,288	566,511	676,205	812,031	1,002,293	1,204,213
North Carolina	10,000	14,677	21,000	33,904	55,950	72,313	109,017	140,860	148,627	182,550	246,812
North Dakota	15,187	17,347	24,908	40,446	62,993	71,678	82,885	90,840	92,644	99,052	109,266
Ohio	86,156	122,504	181,332	252,431	346,772	412,775	511,031	621,390	720,634	858,716	1,069,100
Oklahoma	[2] 3,000	13,500	25,032	52,718	100,199	121,500	144,500	212,880	221,300	249,659	307,000
Oregon	13,975	16,447	23,585	33,917	48,632	63,324	83,332	103,790	118,198	134,125	165,962
Pennsylvania	80,178	112,854	160,137	230,578	325,153	394,186	482,117	570,164	689,589	829,737	1,043,770
Rhode Island	10,295	12,331	16,362	21,406	37,046	36,218	44,833	50,477	54,608	66,083	76,312
South Carolina	10,000	14,000	15,000	[1] 25,000	38,332	55,492	70,143	93,843	89,836	95,239	127,467
South Dakota	14,457	20,929	28,724	44,271	67,158	90,521	104,628	120,395	119,274	125,241	131,700
Tennessee	[1] 10,000	[6] 19,769	[7] 7,618	[1] 30,000	[1] 48,000	[1] 63,000	80,422	101,852	117,025	135,716	173,365
Texas	[1] 32,000	[1] 40,000	[1] 40,000	[1] 125,000	192,961	251,118	331,310	427,693	467,616	526,238	688,233
Utah	4,000	2,253	9,177	13,507	24,076	32,273	35,236	42,616	47,485	49,164	59,525
Vermont	5,913	8,475	11,499	15,671	21,633	22,553	26,807	31,625	37,265	43,881	52,776
Virginia	9,022	13,984	21,357	35,426	55,661	72,228	94,100	115,470	139,200	168,000	218,896
Washington	24,178	30,253	38,823	60,734	91,337	117,278	148,775	173,920	185,359	210,716	258,264
West Virginia	5,144	6,159	13,279	20,571	31,300	38,750	50,203	80,664	93,940	112,763	157,924
Wisconsin	34,346	53,161	79,741	115,645	158,637	196,253	236,290	293,298	341,841	382,542	457,271
Wyoming	1,584	2,428	3,976	7,125	12,523	16,200	21,371	23,926	26,866	30,637	39,831
Total	1,258,062	1,711,339	2,445,666	3,512,996	4,983,340	6,146,617	7,565,446	9,231,941	10,463,295	12,238,375	15,092,177

[1] Estimated.
[2] Includes reregistrations, but does not include nonresident registrations.
[3] State registrations only.
[4] Cars registered during 1916 only; total in State, approximately 138,000.
[5] Cars registered during 1917 only; total, approximately 160,000.
[6] Total cumulative registrations; annual registration not required.
[7] Cars registered during 1915 only; total, approximately 26,000

TABLE III.—*Summary of total gross receipts from motor vehicle registration fees, licenses, and permits, etc., for years 1913 to 1923.*

States.	1913	1914	1915	1916	1917	1918	1919	1920	1921	1922	1923
Alabama	$83,000	$113,202	$180,744	$203,655	$217,700	$470,274	$541,348.70	$835,178.00	$1,147,265.00	[1] $1,262,800.00	$1,541,017.56
Arizona	27,545	34,077	45,579	73,000	117,643	142,288	164,755.68	192,368.92	195,969.75	216,598.26	281,670.75
Arkansas	17,411	56,420	80,551	150,000	205,176	410,649	500,970.00	591,464.50	856,543.60	1,030,196.60	1,435,090.00
California	75,000	1,338,785	2,027,432	2,192,699	2,846,030	3,524,036	4,468,721.67	5,714,717.40	6,834,089.52	8,384,606.40	10,608,544.00
Colorado	60,833	80,047	120,801	197,795	296,808	379,559	490,432.31	819,872.74	906,059.27	991,677.22	1,126,218.55
Connecticut	316,667	406,623	536,970	768,728	1,080,757	1,285,164	1,516,136.91	1,852,591.00	2,129,861.12	3,567,744.84	4,329,432.16
Delaware	24,735	35,672	55,596	85,249	133,883	232,449	286,333.00	329,980.00	375,469.00	426,377.00	516,209.00
Distriet of Columbia	13,228	20,147	29,396	47,624	55,928	220,753	274,184.00	266,285.00	209,583.00	353,726.50	357,918.00
Florida	[2] 6,000	[2] 6,736	60,000	127,176	[3] 170,000	345,775	401,317.40	554,695.14	734,845.50	1,538,342.26	1,963,065.99
Georgia	12,000	104,575	125,000	154,735	229,653	331,816	429,848.00	1,919,338.92	1,705,941.24	1,830,047.61	2,156,406.08
Idaho	35,160	58,580	121,259	213,758	412,641	576,555	729,702.94	882,034.51	841,212.93	812,943.72	914,014.58
Illinois	507,629	699,725	924,906	1,236,566	1,588,835	2,764,330	3,262,714.00	5,915,700.17	6,803,556.21	7,882,482.02	9,653,796.04
Indiana	150,345	432,309	587,318	817,285	1,096,159	1,293,128	1,558,740.50	2,029,694.00	2,422,227.00	2,999,588.50	3,693,715.00
Iowa	787,411	1,040,136	1,533,054	1,776,170	2,249,655	2,547,596	3,077,445.81	7,507,202.08	7,719,127.47	7,923,388.06	8,827,062.99
Kansas	186,066	268,471	387,588	585,762	830,878	978,837	1,150,000.00	1,419,345.50	1,400,000.00	[1] 3,100,000.00	3,435,606.00
Kentucky	52,000	85,883	117,117	184,741	287,314	402,250	565,520.21	815,549.31	1,771,887.02	2,140,444.31	2,678,732.89
Louisiana	[3] 10,000	[3] 12,000	75,600	112,000	166,835	240,000	306,000.00	390,000.00	453,276.00	1,756,226.42	2,191,240.81
Maine	138,509	192,542	268,412	363,562	491,696	570,171	685,570.25	818,755.50	1,004,750.25	1,417,507.57	1,660,268.17
Maryland	150,000	268,231	386,565	565,302	807,395	1,189,984	1,776,410.22	2,124,924.84	2,460,162.04	2,824,843.91	3,536,955.20
Massachusetts	764,154	923,961	1,235,724	1,602,958	1,969,994	2,184,821	2,667,853.85	3,860,231.70	4,717,389.30	5,685,527.05	6,989,633.25
Michigan	190,329	[4]	373,833	1,739,344	2,471,271	2,875,266	3,719,433.39	5,754,900.96	6,751,924.51	8,385,022.17	10,500,786.05
Minnesota	40,000	132,398	[3] 160,540	82,469	100,000	1,076,811	218,469.50	143,794.50	5,672,424.61	6,543,685.77	7,316,772.03
Mississippi	24,735	51,146	76,700	175,000	250,000	335,000	400,000.00	800,000.00	751,946.63	1,179,803.00	1,077,616.22
Missouri	173,510	235,873	323,289	439,315	617,942	1,394,762	1,725,076.70	2,111,696.85	2,505,353.90	3,512,182.97	4,016,383.60
Montana	12,000	27,000	33,120	52,768	290,936	350,914	407,848.00	416,245.00	594,520.50	619,899.50	729,621.50
Nebraska	26,000	34,325	[3] 183,000	311,334	451,303	536,897	304,450.55	2,800,000.00	2,824,811.25	3,031,699.93	3,353,175.32
Nevada	3,323	4,331	7,875	20,116	31,166	31,083	37,550.75	103,318.33	102,800.00	120,937.73	153,888.10
New Hampshire	152,834	185,288	257,776	344,434	425,305	509,335	599,621.25	654,702.04	876,322.14	1,246,098.46	1,571,326.96
New Jersey	661,446	814,536	1,062,923	1,406,806	1,923,164	2,431,757	2,931,902.15	3,503,936.76	3,974,063.75	6,251,418.50	7,653,780.37
New Mexico	15,084	19,663	29,625	47,865	80,843	105,631	111,150.00	200,000.00	198,632.77	243,813.61	295,000.00
New York	1,275,727	1,529,852	1,991,181	2,658,042	4,284,144	4,945,298	5,984,659.50	8,863,250.59	10,288,858.25	12,736,364.37	19,862,441.52
North Carolina	60,000	89,580	123,000	206,101	321,923	394,739	1,313,950.73	1,785,000.00	2,259,240.43	2,715,331.58	3,728,044.72
North Dakota	41,961	55,964	79,245	125,283	211,536	471,429	636,842.40	691,500.00	683,052.45	698,931.70	760,852.45
Ohio	457,538	685,457	984,622	1,286,405	1,766,427	2,125,426	2,593,000.00	6,400,000.00	6,894,159.73	7,888,992.38	9,662,370.29
Oklahoma	3,000	13,500	154,892	555,011	853,659	1,102,380	1,178,130.27	2,500,000.00	2,619,713.49	2,729,169.15	3,217,770.84
Oregon	56,873	77,592	108,881	146,232	196,787	461,422	602,239.00	2,085,168.50	2,334,931.25	3,340,519.58	4,069,609.40
Pennsylvania	841,062	1,185,039	1,665,276	2,325,057	3,268,025	4,048,186	5,090,921.00	8,090,873.04	9,470,174.31	12,575,380.56	15,844,303.80
Rhode Island	129,851	157,020	206,440	264,737	346,117	385,608	477,223.25	531,462.75	848,723.59	1,139,742.77	1,286,659.47
South Carolina	10,000	14,000	15,000	10,000	113,557	300,217	389,034.68	527,868.13	741,114.79	734,856.18	902,608.69
South Dakota	89,170	125,000	[3] 180,000	140,746	210,592	282,742	322,340.50	784,000.00	720,587.00	743,232.00	1,130,959.27
Tennessee	[3] 9,000	39,538	[3] 34,000	186,953	322,200	390,000	585,181.95	1,215,776.04	1,387,870.10	1,592,230.14	2,049,653.27
Texas	16,000	20,000	20,000	20,000	858,978	2,039,589	2,624,334.29	3,510,355.97	3,806,395.25	4,261,488.67	5,441,508.59
Utah	3,000	4,852	[3] 60,000	93,494	170,707	229,203	291,325.96	350,933.29	441,359.88	729,455.00	430,104.72
Vermont	111,460	154,267	218,480	297,992	363,541	398,856	460,190.87	555,422.38	668,288.50	781,982.35	938,860.30
Virginia	83,611	120,814	176,875	271,266	518,566	684,636	900,000.00	1,822,736.16	2,021,146.09	2,467,346.93	3,200,161.66
Washington	48,356	60,506	238,717	350,052	519,526	875,391	2,325,323.53	2,828,896.10	3,140,730.74	3,291,671.70	3,898,597.77
West Virginia	40,000	60,648	128,952	198,436	359,339	447,705	1,008,083.31	1,280,193.28	1,250,525.82	1,936,079.29	2,608,508.37
Wisconsin	190,770	293,580	431,977	615,721	861,278	2,076,701	2,502,852.00	3,127,073.00	3,671,645.50	4,088,570.00	4,958,933.55
Wyoming	7,920	12,140	19,880	35,625	57,421	80,000	102,114.50	267,179.35	288,121.88	316,849.50	414,096.39
Total	8,192,253	12,382,031	18,245,711	25,865,369	37,501,233	51,477,419	64,697,255.48	102,546,212.25	122,478,654.33	152,047,823.74	188,970,992.24

[1] Approximate. [2] State registrations only. [3] Estimated. [4] Registration law held unconstitutional.

TABLE IV.—*Gasoline taxes, 1923.*

State.	Total gross receipts.	Amount applicable to highway work by or under supervision of State highway department.	Tax in cents per gallon on Jan. 1, 1923.	Tax in cents per gallon on Jan. 1, 1924.	Date on which change in rate of tax became effective.
Alabama	$1,133,085.49			2	Mar. 1.
Arizona	474,123.04	$118,530.76	1	3	June 9.
Arkansas [1]	1,219,198.75	301,094.76	1	4	Jan. 1, 1924.
California	2,518,893.00	1,259,446.00		2	Sept. 30.
Colorado	846,353.12	402,017.73	1	2	Aug. 1.
Connecticut	880,222.70	880,222.70	1	1	
Delaware [2]	88,579.28	88,579.28		2	Jan. 1, 1924.
Distriet of Columbia					
Florida	1,641,042.25	1,150,355.99	1	3	July 1.
Georgia	1,502,503.49	247,666.55	1	3	Oct. 1.
Idaho	396,487.18	396,487.19		2	Apr. 1.
Illinois					
Indiana	2,906,428.25	2,514,755.83		2	June 1.
Iowa					
Kansas					
Kentucky	680,435.30	680,435.30	1	1	
Louisiana	754,437.85	754,437.85	1	1	
Maine	286,076.97	285,839.84		1	July 7.
Maryland	688,304.02	688,304.02	1	2	Jan. 1, 1924.
Massachusetts [3]					
Michigan					
Minnesota [4]					
Mississippi	467,855.53	187,142.21	1	1	
Missouri					
Montana	441,249.10	75,877.10	1	2	July 1.
Nebraska					
Nevada	115,843.24	60,000.00		2	Mar. 20.
New Hampshire [5]	163,064.64	161,823.10		2	Jan. 1, 1924.
New Jersey					
New Mexico	165,000.00	156,750.00	1	1	
New York					
North Carolina	2,909,904.74	[7] 2,900,000.00	1	3	Apr. 1.
North Dakota	461,081.71		1	1	
Ohio					
Oklahoma	599,000.00	599,000.00		1	July 1.
Oregon	1,958,141.37	1,885,421.15	2	3	May 24.
Pennsylvania	5,491,522.66		1	2	July 1.
Rhode Island					
South Carolina	1,511,452.56	411,327.78	2	3	Mar. 23.
South Dakota	624,692.44	[7] 565,000.00	1	2	
Tennessee	812,356.68	801,502.36		2	Apr. 1.
Texas	1,215,623.36	911,717.52		1	June 15
Utah	404,085.81	106,902.75		2½	Mar. 8.
Vermont	168,172.81	168,172.81		1	Apr. 1.
Virginia	1,556,920.99	[8] 1,037,947.32		3	June 27.
Washington	1,225,149.66	1,225,149.66	1	2	Jan. 1, 1924.
West Virginia	[9] 366,490.00	366,490.00		2	July 27.
Wiseonsin					
Wyoming	140,161.62	140,161.62		1	Mar. 1.
Total	36,813,939.61	21,528,559.18			

[1] One cent from Jan. 1 to Apr. 1; 3 cents Apr. 1 to Dec. 31, inclusive.
[2] One cent from Apr. 22 to Dec. 31, inclusive.
[3] Act passed by legislature, but referendum invoked. Election November, 1924.
[4] Constitutional amendment permitting a tax on gasoline to be voted on November, 1924.
[5] To Dec. 1, 1923.
[6] Tax of 1 cent per gallon effective July 1 to Dec. 31, inclusive.
[7] Approximate.
[8] One-third of total receipts returned to counties for county highway work.
[9] Collections from July 27, when tax became effective, to Nov. 1.

THE BRICK ROADS OF FLORIDA.

OBSERVATIONS OF THE BEHAVIOR OF BRICK SURFACES LAID ON CONFINED SAND SUBGRADES.

By C. A. HOGENTOGLER, Highway Engineer, U. S. Bureau of Public Roads.

Brick roads in Florida are generally laid directly upon the sand subgrade. The brick are filled with sand, cement grout, or a bituminous filler, and are retained at the sides of the road by cypress, vitrified brick, or concrete curbs. The shoulders are sand, shell, clay, or limerock.

Brick pavements of this type have practically no "beam strength." Traffic loads are supported by the confined sand of the subgrade, the brick surface serving chiefly as a wearing course.

The cement-grout filled pavements offer less tractive resistance and more resistance to deterioration than the sand-filled roads. They distribute the wheel loads over a wider area of subgrade, but when they do break up, as they do when laid directly on the sand subgrade, the relaying value of the brick is much less than those of sand-filled pavements. If in the future this type of surface is laid with adequate base and shoulders and provision for expansion, it should make a very satisfactory pavement.

Sand-filled pavements in service from 8 to 14 years, receiving little or no intelligent maintenance, have 70 per cent of their surface functioning as fair and good road. The remaining 30 per cent, while in need of reconstruction, has a reconstruction value of 80 per cent of the new construction cost. Pavements of this type are believed to have been economical and well adapted to a State in which uncertain development precluded a forecast of future traffic conditions.

Sand is ineffective as a filler. It is not waterproof; it offers but little frictional resistance to the movement of the brick; and it can not be retained in place.

Bituminous fillers have desirable waterproofing and plastic properties. They do not reduce the salvage value of the pavement.

Indications are that limerock dust, sand mixed with limerock dust or clay, or sand treated with a light oil, would make satisfactory fillers.

Indications are that the brick roads can be strengthened to accommodate heavy traffic or to hold up in locations where good drainage is impracticable by laying a base course of compacted limerock which will provide additional confinement of the sand subgrade. Such bases are not like rigid slabs which have "beam strength"; their strength does not lie in the bond between the individual stones as in the macadams; their chief value lies in their ability to remain at all times in contact with the sand subgrade. They have little supporting power in themselves, but they do make available the maximum support of the subgrade. They also prevent loss of sand through the crevices between the brick.

Shoulders of limerock or other material capable of providing side support for the pavement are more necessary than thick bases.

Florida State Highway Commission reports indicate a maximum daily traffic over such roads amounting to 200 motor trucks on the two-way road between Lakeland and Tampa and 157 on the narrow one-way road between Sanford and Kissimmee. The number of trucks averaged from 10 to 30 per cent of the total number of vehicles.

It is important to note that practically all of the Florida road traffic is carried on pneumatic tires, for it is quite probable that equally good results as herein reported would not have been secured had the heaviest trucks operated on solid tires. It should also be noted that Florida traffic is comparatively light. The gross load limitation is 16,000 pounds on pneumatic tired vehicles and 8,000 pounds on vehicles having solid tires

RECENT highway research has developed considerable information concerning the strength and wearing properties of rigid surfaces possessing what is called beam strength when such surfaces are laid on subgrades of questionable bearing value.

This article presents the results of a number of observations of a kind of road which is the exact antithesis of such construction. The brick roads of Florida have little or no beam strength, but they are laid on a subgrade which, when properly confined, has high supporting value.

The unpaved sand roads are exceedingly unstable. Their tractive resistance is high. Yet the surfacing of such roads with a pavement of brick, in many instances without other filler than the sand itself, and the installation of a curb along the edges of the brick surface converts them into excellent thoroughfares which have been a source of great satisfaction to thousands of winter visitors to the "Land of Easter."

The Florida sand soils lack stability because of the poor gradation of the sand particles and their low clay content. When properly compacted and confined, however, they offer high resistance to volume change, a fact which, with the absence of frost, accounts for their extensive use as bases for brick pavements. The characteristic instability of the soil is indicated by Figure 1, which shows the result of the passage of a few vehicles. That sand-filled brick surfaces have little or no beam strength is apparent in Figure 2, which shows the separation of brick and loss of filler on State route No. 1 between Jacksonville and Lake City. This surface, which is representative of the Florida construction, has withstood traffic for 14 years, including the war traffic of truck trains, gun mounts, and carriages. While this surface is not in perfect condition, its principal defects are directly traceable to lack of stability, the one undesirable characteristic of the sand subgrade. It is clear from a study of these roads that when subgrades have stability and high supporting power and are subject to little volume change the chief function of the surface is that of a wearing course, capable of utilizing the maximum support offered by the subgrade.

STAGES OF DETERIORATION NOTED.

The rural brick surfaces of Florida are usually 9 or 15 feet wide and are supported by cypress, vitrified brick, or concrete curbs. Sand, cement-grout, and bituminous fillers are used, and the shoulders consist of sand, shell, clay, or limerock.

The deterioration of sand-filled surfaces is first indicated by a more or less uniform settlement along the curbs, accompanied in some instances by transverse separation of the brick.

The second stage of deterioration is indicated by well-defined grooves or edge depressions, loss of filler, transverse and longitudinal movement of the brick, and, in some cases, slight unevenness of surface. Looseness causes the bricks to rattle during the passage of vehicles, and broken bricks are often displaced.

The third stage of deterioration or failure of sand-filled pavements is indicated by excessive rutting, separation and displacement of bricks, and unevenness of surface.

The progress of deterioration in bituminous-filled surfaces could not be determined because of the newness of this type of construction in Florida. All the observed surfaces of this type, however, showed separation of brick, accompanied in some instances by slight grooving. The latter was found principally on curves.

FIG. 1.—Graded natural Florida soil road. State Route No. 2, south of Crescent City.

In cement-grout-filled surfaces the first signs of deterioration noted were cracks, which in some cases, were so frequent as to divide the pavements into small blocks. These cracks, which generally were found only on close inspection, while they did not change perceptibly the contour of the surface, did change the pavement from a rigid to a more or less flexible top.

When the cracks became noticeable due to slight feathering of the edges or separation of the pavement blocks, and when these blocks were not displaced vertically, a second stage of deterioration was indicated.

When excessive cracking separated practically every brick, or when individual areas became so displaced that traffic was seriously hindered, the third stage of deterioration or failure was indicated.

Surfaces showing only the first indications of deterioration offered all of the advantages that could be expected from the best paved roads.

In the second stage of deterioration, though the pavements showed plainly the effects of climate and traffic, and in many cases indications of gross neglect, they offered but little inconvenience to traffic and in general functioned as first-class roads. One of the interesting features of the narrow, sand-filled surfaces was that even deep grooves did not as a rule impair the smooth riding qualities of the pavement. They inconvenienced traffic to some extent, however, when vehicles passed each other.

FIG. 2.—State Route No. 1 between Jacksonville and Lake City, showing lack of "beam strength."

The roughness of surfaces in the third stage of deterioration seriously inconvenienced traffic which, to prevent accident, was forced to move at considerably reduced speed.

EXAMPLES OF PAVEMENTS IN FIRST STAGE OF DETERIORATION.

The following pavements were representative of those which showed no greater deterioration than that described as the first stage.

Park Avenue, Sanford, a view of which is shown in Figure 3, is a residential street constructed in 1906, with sand-filled, repressed brick, laid on edge. The pavement was still giving excellent traffic service.

The Boulevard, DeLand, is representative of 4 miles of cement-grout-filled, wire-cut-lug pavement constructed in 1916–17, at a cost of $1.65 per square yard. Ranging from 24 to 44 feet in width, these pavements were laid in business as well as residential streets. The main business street carried also the traffic of State route No. 2, Orlando to DeLand. The principal defects noted were one blow-up at a street and grade intersection, one longitudinal crack caused by the fail-

FIG. 3.—Park Avenue, Sanford—A sand-filled pavement in the first stage of deterioration, still giving excellent traffic service.

FIG. 4.—Bituminous-filled relaid brick surface between Jacksonville and Atlantic Beach, giving good service though in the first stage of deterioration.

ure of a fill, and several cracks adjacent to a railroad crossing.

The bituminous-filled pavement between Jacksonville and Atlantic Beach, a view of which is shown in Figure 4, is 18 feet wide. It was relaid in 1919 with repressed bricks from the sand-filled pavement constructed in 1909. When observed it showed slight transverse separation of the brick and indications of grooving, especially on curves.

A section of State route No. 1, in Jacksonville, a cement-grout-filled, wire-cut-lug surface, shown in Figure 5, was in excellent condition after four years of service.

A section of State route No. 2, between Orlando and Sanford, was another cement-grout-filled pavement of 3½-inch wire-cut-lug brick, the surface being 8 feet wide. It was laid in 1917 with concrete curbs and showed behavior typical of narrow pavements of this type. In a length of 7.6 miles 16 breaks were noted. Five of these were blow-ups which occurred at grade apexes. Two breaks were adjacent to railroad crossings and five occurred at curves. Figure 6 shows a break near a curve in this pavement. Water can be seen in the crack between the curb and the shoulder. The surface had been pushed outward 5 inches from the inside curb on a 90° turn, and about 2½ inches on a 30° turn. The outside curb had been broken off in the latter case.

FIG. 5.—Cement-grout-filled brick surface, State Route No. 1, near Jacksonville, giving excellent service after four years, though in the first stage of deterioration.

THE SECOND STAGE OF DETERIORATION ILLUSTRATED.

Pavements in the second stage of deterioration were represented by the following:

The section of State route No. 4, Jacksonville to Waycross, 15 feet wide, consisting of sand-filled, repressed brick, laid on the side, with 4 by 10 inch concrete and brick curbs, was constructed in May, 1910, at a cost of $1.48 per square yard. It is about 5 miles long, beginning 3½ miles from the city limits of Jacksonville. As shown in Figure 7, a good, even riding surface was presented in spite of the fact that the curbs were pushed out of line, that a considerable transverse separation of the brick existed, and that along the side edge there were depressed areas which held water after rains. Increased settlement was noted on fills through cypress swamps. Relaying of

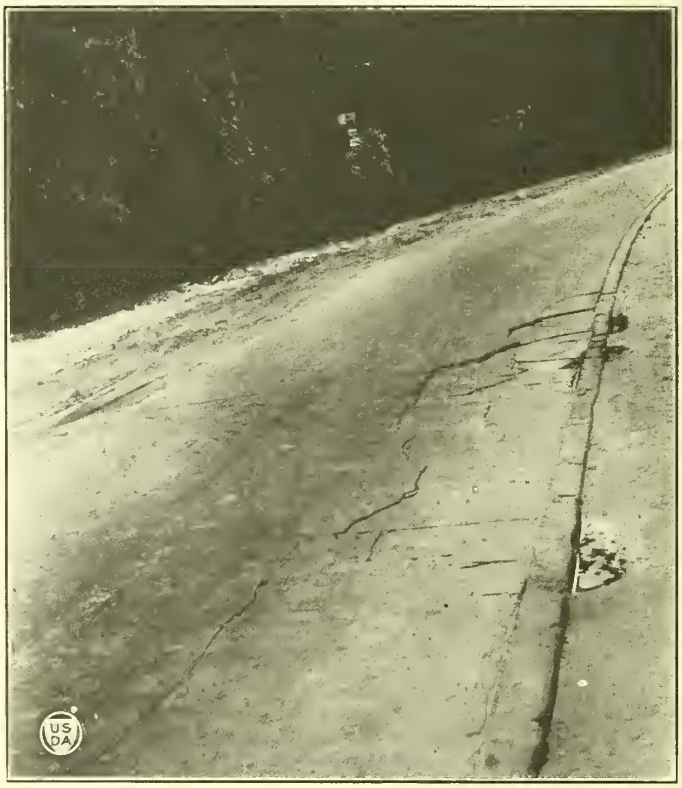


FIG. 6.—Break on curve of cement-grout-filled surface—State Route No. 2, between Orlando and Sanford.

uneven areas over culverts and salamander holes and mounds, caused probably by stumps in the subgrade, and application of additional filler would make this a first-class road.

Two sections of the above road each one-half mile in length were constructed with depressed centers, as shown in Figure 8. More or less uniform depression of brick along the edges was noted, as well as a slight depression several feet from the left-hand edge. The curbs were pushed out of line, but the sand filler seemed to be intact. These depressed-center sections were constructed in 1909, after serious washing of the shoulder of the previous pavement. Although water had been flowing down the middle of the road for something like 14 years, these sections presented very good riding surfaces. They also eliminated the trouble found on the majority of Florida roads—that of water standing in depressions along the curb.

FIG. 7.—Sand-filled brick surface, north of Jacksonville. In the second stage of deterioration but still giving good service.

Figure 9 illustrates the loss of filler and the brick movement characteristic of the second stage of deterioration in a sand-filled pavement.

The section of State route No. 4, south of St. Augustine, is 16 feet wide. It was graded in 1908, paved in 1909–10 with sand-filled, repressed brick laid on the side, and retained by brick curbs. This road carries practically all of the tourist traffic between north and south Florida, including busses which have a capacity of 35 passengers and run at a speed of about 35 miles an hour. In some places this road has grass shoulders (fig. 10), and considerable grass grows in the filler between the brick. There is more or less depression of the sides in all sections, which is particularly noticeable in the lower view, Figure 10. In addition to the depression of the sides a longitudinal movement of the brick amounting to about three-fourths of an inch was observed.

Representative also of the second stage of deterioration were the sections of sand-filled brick surfaces 9 feet wide between St. Augustine and Hastings. The typical condition of these pavements is shown by Figure 11.

FIG. 8.—Section of sand-filled brick pavement with depressed center—State Route No. 4—north of Jacksonville. Although water has flowed down the middle of this road for 14 years it still presents a good riding surface.

THE THIRD STAGE OF DETERIORATION.

The following surfaces were representative of those in the third stage of deterioration:

A section of sand-filled brick surface with wooden curbs between Orlando and Sanford, a view of which is shown in Figure 12. In this pavement, which was laid in 1916, a very uneven surface was caused by excessive brick movement.

Orange Park Road, passing the United States Government reservation, south of Jacksonville. The unevenness and brick separation in this sand-filled surface, which was constructed in 1910-11, 15 feet wide with brick curbs, are shown in Figure 13.

Sections of narrow, sand-filled brick surface between Hastings and Ormond Beach, a typical view of which is

FIG. 9.—Section of sand-filled surface between Jacksonville and Atlantic Beach, showing movement of brick and loss of filler.

shown in Figure 14. This road was in worse condition than any other seen in Florida. In some places the concrete curbs were 1 foot out of line. The base and shoulders consisted of typical "ball-bearing" sand, and water stood along the sides 6 to 18 inches from the surface.

TRAFFIC SERIOUSLY INCONVENIENCED ON ONLY 25 PER CENT OF PAVEMENTS INSPECTED.

Exclusive of city streets, about 300 miles of brick pavement were inspected. About 35 per cent of this length consisted of excellent surfaces, which either showed no signs of deterioration or in which brick movement, settlement, or cracking had not developed to an extent requiring immediate repair except in small, isolated areas. About 40 per cent consisted of roads which showed marked loss of filler, appreciable cracking, brick movement, grooving, and spread of curbs. Some of these pavements retained remarkably smooth

riding qualities, while in others, such unevenness as existed, inconvenienced traffic but little. The remaining 25 per cent consisted of surfaces on which traffic was seriously inconvenienced or hindered by marked brick movement, loose bricks, peaked centers, nonuniform grooving or displacement of individual parts.

The above percentages are based on a classification of whole sections. If the general characteristics of a section were such as to place it within the third class the length of the whole section was used in determining the percentage. If considered foot by foot, the first-class percentage would be increased, since there were short lengths of excellent surface in many sections whose general condition indicated second and third stages of deterioration.

INADEQUATE SIDE SUPPORT THE CAUSE OF MAJOR DEFECTS.

It is apparent that inadequate confinement of the sand subgrade was responsible for the major defects in the Florida pavements, and also that, in the main, this inadequate confinement was due to insufficient side support. Defects not attributable to the above cause were initial settlement, cracking and breakage, and unevenness found in lowland and swamp locations.

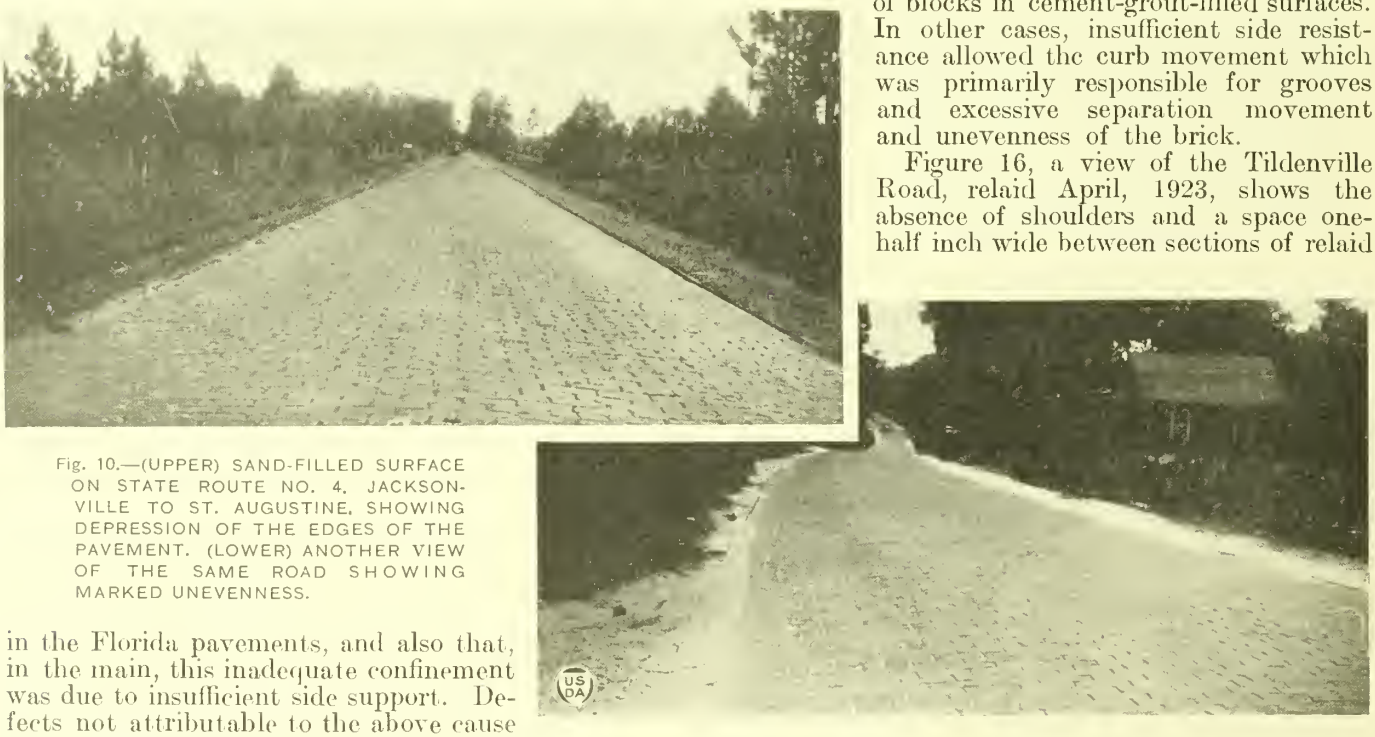


Fig. 10.—(UPPER) SAND-FILLED SURFACE ON STATE ROUTE NO. 4, JACKSONVILLE TO ST. AUGUSTINE, SHOWING DEPRESSION OF THE EDGES OF THE PAVEMENT. (LOWER) ANOTHER VIEW OF THE SAME ROAD SHOWING MARKED UNEVENNESS.

Initial settlement in both sand and bituminous-filled surfaces was probably due to settlement of the sand subgrade and can not be eliminated in this type of construction. That even a nonrigid base course will not entirely eliminate this defect is indicated by Figure 15, a view of a sand-filled, repressed brick surface, with concrete curbs, shell base and shoulders, laid between Jacksonville and St. Augustine in 1916. This minor defect can be reduced to a minimum by adequately compacting the subgrade prior to laying the base and by sufficiently rolling the bricks before applying the filler. Retention of rain water on the surface, the chief inconvenience caused by slight settlement will be avoided by laying the bricks so that their tops are slightly above the top of the curb.

Cracking and breakage in cement-grout-filled pavements were due to surface movement caused by expansion and the adjustment of the rigid slabs to their new positions. These defects can not be eliminated in this type of pavement.

Unevenness of surfaces in lowland and swamp locations was attributed both to the character of the sand and the presence of water. The sand in these locations is excessively fine and especially when wet has considerably less stability than normal soil. There was some indication also that this type of sand works up through the crevices between the brick and is either washed or blown away. The injurious effect of water in all types of sand was plainly evident, although in some cases, very good road surfaces were found in locations where elimination of water was impracticable.

Where good drainage existed, the effect of curbs and shoulders on the behavior of the pavements was very apparent. In some cases, total absence of shoulders allowed the subgrade to be washed away. Neglect of this kind was undoubtedly responsible for nonuniform settlement in sand-filled, and displacement of blocks in cement-grout-filled surfaces. In other cases, insufficient side resistance allowed the curb movement which was primarily responsible for grooves and excessive separation movement and unevenness of the brick.

Figure 16, a view of the Tildenville Road, relaid April, 1923, shows the absence of shoulders and a space one-half inch wide between sections of relaid curbing. Within a radius of 18 inches around the joint the sand filler as well as about one inch of the subgrade had been washed away. When this amount of displacement occurs in several months, one can easily conjecture what would happen in a period of years.

TYPICAL DEFECTS OF A CEMENT-GROUT-FILLED SECTION.

A cement-grout-filled, wire-cut-lug surface between Orlando and Bithlo showed very clearly the influence of shoulders on the behavior of pavements. The first 100 feet, beginning at the end of East Colonial Street, Orlando, had good sand shoulders flush with the pavement and was practically free from cracks. Slightly beyond this, there were indications that water flowed along the north side, and coincident with this condition a longitudinal crack 18 inches from the north edge of the pavement was noted. Some distance farther

along the route poor drainage, mucky sand soil, inefficient shoulders, and practically no curbs were encountered, and, as would be expected, considerable breakage of surface. Figure 17 shows the absence of shoulders and the remains of the old cypress curb. Although the pavement had little support this section was in very good condition. Figure 18 shows the breakage which occurred where shoulders were absent and water stood within 6 inches of the top of the surface. The displacement of surface shown in Figure 19 could not have occurred had the sand subgrade not been removed, and since the shoulders adjacent to this and to several other similar breaks were from 6 to 8 inches below the surface, water probably was the cause of the settlement.

In 2.2 miles there existed several depressed areas such as the one shown in Figure 19, several locations where edge breaks occurred, such as the one shown in Figure 18, and four blow-ups extending from 50 to 150 feet each. The combined length of all breaks was somewhat less than 10 per cent of the length of the surface. The remainder of the road, although considerably cracked, was in the good riding condition shown in Figure 17. Traffic failures were noted only where no shoulders existed. The breakage in this road occurred between April and July, 1923, during which time about 5,000 loads of limerock were hauled over it. Figure 20 shows one of these loads consisting of three cubic yards of rock on the truck and two on the trailer.

In passing from the cars to the road these trucks passed over Mills and East Colonial Streets in Orlando, and the indications were that they traveled in the same path on each trip. Both street surfaces were sand-filled, repressed brick, laid flat. In some places on the former, which had been in service for about one year, slight indications of settlement in the wheel tracks were found on close examination. In the latter, which had been in service about two years, the inside

FIG. 11.—Grooves in a narrow sand-filled brick surface between St. Augustine and Hastings.

path was over a gas or water main trench and there was settlement ranging in depth from one to two inches. Rigid examination, however, failed to disclose any indication of settlement in the outer wheel path. At the intersection of the two streets, appreciable settlement was noted where the wheel paths crossed a gutter which carried surface water during rains.

FIG. 12.—Sand-filled brick surface with wood curbs, between Orlando and Sanford. This surface is in the third stage of deterioration. Note the settlement and separation of the brick.

VALUE OF SIDE SUPPORT ILLUSTRATED.

The effect of side support in sand-filled pavements was illustrated by the road between Orlando and Kissimmee, which was constructed 9 feet wide, with 4 by 10 inch curbs in 1916. On the first section, between Orlando and Pine Castle, a distance of 3 miles, clay shoulders were laid immediately after construction; on the second section 2 miles south from Pine Castle the shoulders were laid with rock between 18 months and 2 years later; and along the third section, rock shoulders were laid partly last year and partly this year. That portion where shoulders were not laid until this year was more rutted than any other part of the Orlando-Kissimmee Road. While rutting and separation seemed more pronounced in the second than in the first section, it was only in the third section that many areas were relaid, these areas being principally where the road was subjected to action of surface water.

Displacement of the sand subgrade due to vertical movement of curbs was noted on the Orlando-Winter Garden Road and was probably responsible for the failure shown in Figure 21. Only two conditions of this kind were found in a length of 3.2 miles of this sand-filled, repressed-brick surface with concrete curbs, which was laid in 1916. Bulging of the curbs began soon after construction and was caused by expansion. Until spaces were found under the curb, the pavement was thought to have settled. This upward movement of the curbs was increased by pressure of the subgrade sand, which was forced into the spaces beneath them, causing variation in the curb line and settlement of adjacent bricks. Although deeply grooved as shown in Figure 22, a smooth-riding surface existed. This surface is now being relaid, 15 feet wide.

The Orlando-Kissimmee Road referred to above is part of the Sandford-Plant City Road, which, according to a traffic census taken by the Florida State highway commission, January to June, 1923, carried heavier truck traffic than any of the other roads observed. Because all parts of this road received the same intelligent maintenance, the behavior of its various surfaces was considered indicative of their adequacy for Florida conditions. Considering only those surfaces 9 feet wide which had been in service about eight

years, it was noted that of the cement-grout-filled surfaces, all of which had 4 by 10 inch concrete curbs, 95 per cent were in class 1, 2½ per cent in class 2, and 2½ per cent, due to failure of the sand fill, were in class 3. Of the sand-filled surfaces, those which had 4 by 10 inch concrete curbs and limerock shoulders laid at the time of construction were in class 1, those which had 4 by 10 inch concrete curbs and sand shoulders were in class 2, and those with wood curbs as well as a short section through a swamp with concrete curbs were in class 3. Short sections of the sand-filled surfaces with concrete curbs and sand shoulders, especially at locations subjected to water action, had been relaid.

RELAYING VALUE OF SAND-FILLED SECTIONS HIGH.

Apparently the cement-grout-filled offered great resistance to deterioration than the sand-filled surfaces. The present value of the surfaces, however, depends not alone on their condition, but also upon the relaying value of the brick. This relaying value is of special importance in Florida since, at 1923 prices, the cost of brick was about 73 per cent of the total cost of the surface. There was but little salvage value in cement-grout-filled pavements, while in sand-filled pavements there was considerable.

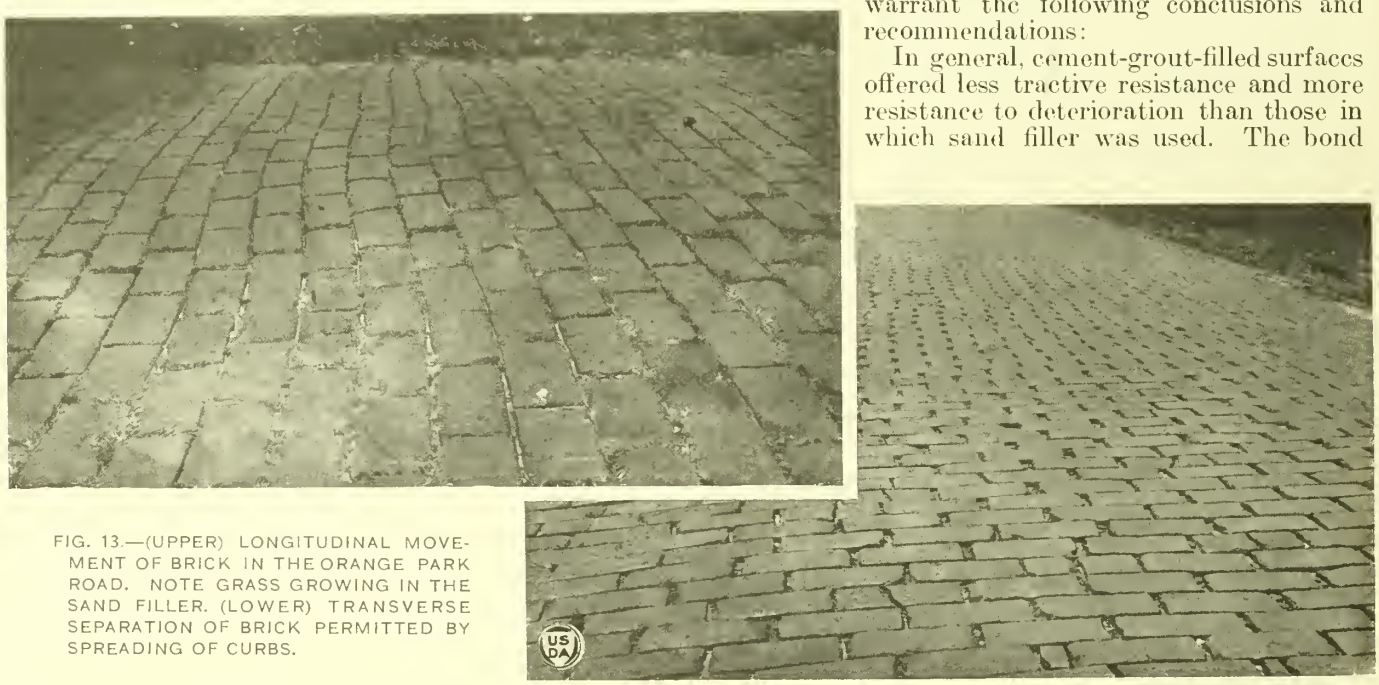


FIG. 13.—(UPPER) LONGITUDINAL MOVEMENT OF BRICK IN THE ORANGE PARK ROAD. NOTE GRASS GROWING IN THE SAND FILLER. (LOWER) TRANSVERSE SEPARATION OF BRICK PERMITTED BY SPREADING OF CURBS.

On a sand-filled section of the Jacksonville-Atlantic Beach Road, laid in 1909 and relaid in 1919, the breakage of brick was less than 1 per cent. On one contract in Orange County 650 square yards of new brick were required in relaying 90,000 square yards of surface originally constructed in 1916. When relaid without disturbing the curbs, a greater percentage of new brick was required because of increased pavement width. Owing to the very small breakage, however, practically the entire cost of the brick can be considered as a permanent investment. The conclusion drawn from the observations was that even when a sand-filled pavement had deteriorated to a condition characteristic of the third stage, at present rates about 70 per cent of the initial investment still remained, the remaining 30 per cent representing the price of the service afforded by the road. When, however, failure occurs in a pavement of low salvage value, such as a cement-grout-filled brick, almost the entire first cost is lost. This pertinent fact must be given proper consideration in the selection of a pavement.

The high salvage value of sand-filled brick was indicated also by the difference between bids for new and relaid pavements. This difference indicated a relaying value of about $1.95 per square yard, or about 80 per cent of the total cost of new construction. It is interesting to note that because of present high prices, the indicated relaying value ($1.95) of bricks in the Orlando-Winter Garden Road was 40 cents greater than the cost ($1.55) per square yard of laying the surface seven years ago.

CONCLUSIONS AND RECOMMENDATIONS.

It is believed that the foregoing descriptions and discussion of the behavior of Florida brick pavements warrant the following conclusions and recommendations:

In general, cement-grout-filled surfaces offered less tractive resistance and more resistance to deterioration than those in which sand filler was used. The bond between bricks minimized the danger of separation and, when adequate shoulders existed, required only wooden side forms. Figure 23 shows that cement-grout-filled surfaces conformed to changing subgrade profiles without becoming rough. This type of surface, because of its rigidity, distributed wheel loads over larger areas of subgrade and consequently subjected the soil to vertical pressure of less intensity than other types. It is not believed, however, that this advantage was of sufficient importance to compensate for the breakage caused by blow-ups and the loss of relaying value of the brick which cement-grout filler causes. If in the future this type of surface is laid with adequate base and shoulders and provision for expansion, it should make a very satisfactory pavement.

Because of the high salvage value of the brick and the small cost of relaying, the sand-filled pavements were economical and especially well adapted to a State in which uncertain development precluded a forecast of future traffic conditions. When pavements which have been in service from 8 to 14 years, and which in

that time in many cases have received little or no intelligent maintenance, have 70 per cent functioning as fair and good roads and the remaining 30 per cent, though in need of reconstruction, has a relaying value equivalent to 80 per cent of the new construction cost, such pavements can not be considered as other than satisfactory.

By making a few changes in future construction, however, it is believed that many of the defects at present existing in the flexible brick tops can be considerably reduced or entirely eliminated.

Sand as filler is very ineffective. It is not waterproof; it affords but little frictional resistance to the movement of the brick, and it can not be retained in place. As a rule, the sand-filled pavements observed were but half filled. Indications were that limerock dust, sand mixed with limerock dust or clay, or sand treated with a light oil, would make satisfactory fillers.

Bituminous fillers seem desirable because of their waterproofing and plastic properties, and also since they do not reduce the salvage value of the pavements.

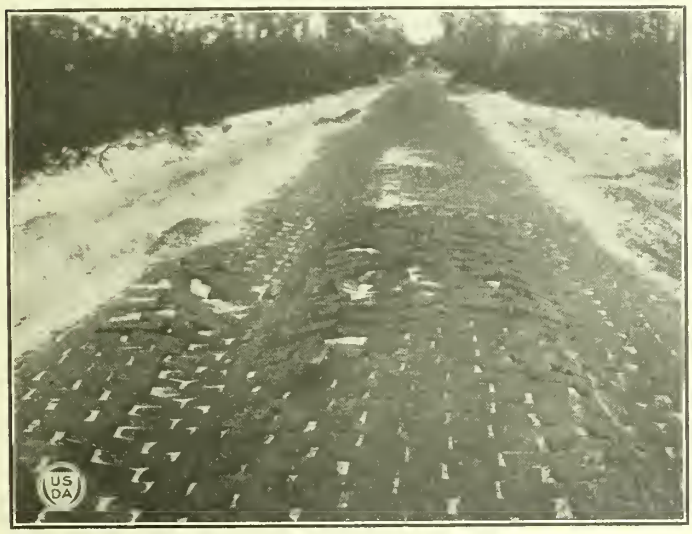


FIG. 14.—Failure of narrow, sand-filled surface between Hastings and Ormond Beach. This is a third-stage pavement.

While it was evident that 4 by 10 inch curbs (concrete or vitrified brick) were incapable of resisting without movement the pressure to which they were subjected, it is believed that, rather than increase the curb size, special shoulders should be laid to furnish the required additional side support.

Sand shoulders, like sand filler, many times meant no shoulders at all. Especially along the narrow roads which carried considerable traffic, sand shoulders could not be retained in place. When covered with grass, they could be retained along the wide roads, but even so, their resistance was insufficient to prevent curb spreading and the accompanying defects.

LIMEROCK A USEFUL MATERIAL FOR SHOULDERS AND BASE.

Indications were that limerock shoulders were very effective and desirable not only on account of their additional resistance but also for the additional effective road width which they furnish.

Where heavy traffic is to be accommodated, and also in all locations where good drainage is impracticable, additional confinement of the soil foundation by means of a base course is believed desirable. For these base courses also the Florida soft limerocks are the most

FIG. 15.—Showing settlement in a narrow, sand-filled surface laid on a shell base This is a section of the Jacksonville-St. Augustine road.

accessible materials. A piece of limerock placed in water will disintegrate, but when thoroughly puddled and rolled, it forms a smooth surface which apparently is little affected by moisture. These surfaces are not hard enough to resist surface wear but seem to have considerable resistance to internal disintegration.

Figure 24 shows a limerock base before rolling. Figure 25 shows the smooth surface obtained by rolling and also its lack of beam resistance. This case on the Tildenville-Black Lake Road was down two weeks when the sand subgrade was washed out, allowing the limerock base to fall. Figure 26 shows a limerock surface which was laid in 1914 by placing the material on the road and allowing traffic to compact it. Figure 27 is a view of the first limerock road constructed in Florida. This section of State-aid Route No. 5 between Ocala and Tampa was properly compacted by a power roller in 1913 and has been maintained with sand. It has resisted both traffic and climate remarkably well and at present is hard and compact and undoubtedly has considerably more slab strength than the base shown in Figure 25.

There is no criterion at present to indicate how thick soft limerock bases should be. They are not like the macadams which depend upon the bond between individual stones, nor are they like rigid slabs which have definite beam resistance. They are more or less

FIG. 16.—View showing absence of shoulder and wide, open curb joint on the Tildenville road.

FIG. 17 (LEFT).—SHOWING GOOD CONDITION OF CEMENT-GROUT-FILLED SURFACE NOTWITHSTANDING THE LOSS OF THE SHOULDERS AND WOODEN CURBS.

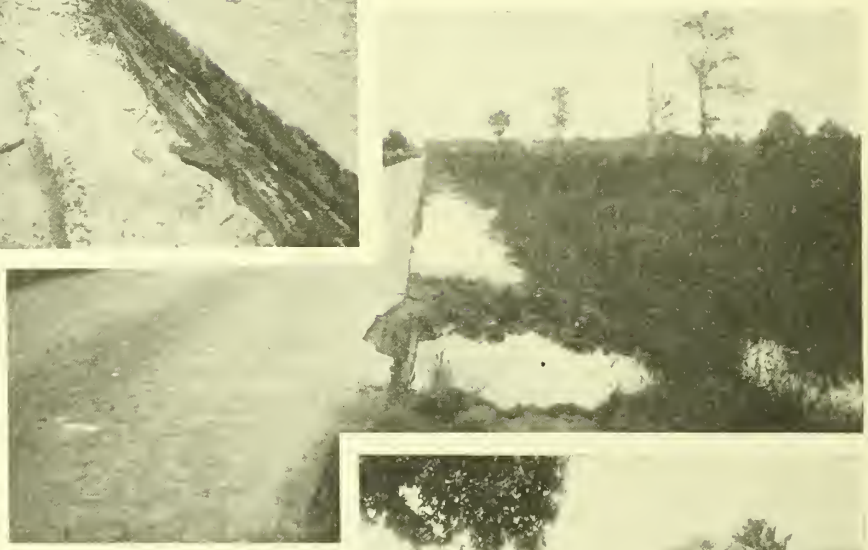


FIG. 18 (RIGHT).—SHOWING HOW EDGES FAILED WHERE WATER STOOD NEAR THE ROAD SURFACE AND SHOULDERS WERE WASHED AWAY.

FIG. 19 (RIGHT).—THIS BREAK COULD NOT HAVE OCCURRED HAD NOT THE SAND SUBGRADE BEEN DISPLACED.
FIG. 20 (ABOVE).—THE KIND OF TRAFFIC WHICH CAUSED THE DAMAGE.

VIEWS ILLUSTRATING THE DETERIORATION OF THE CEMENT-GROUT-FILLED ORLANDO-BITHLO ROAD AND THE CHARACTER OF THE TRAFFIC WHICH PROBABLY WAS RESPONSIBLE FOR THE DAMAGE.

plastic layers whose function is to confine and utilize the maximum bearing value of the soil rather than to support the load.

Made into slabs, cubes, or cylinders, limerock could meet none of the requirements of concrete. Tests of this character, however, do not indicate the relative value of these materials as Florida pavement bases. Concrete because of its rigidity, expansion, contraction, and warping, can not have uniform and continuous subgrade support and therefore must have beam strength to distribute the load. It fails by cracking and shattering. The action and the function of limerock bases are different. While in themselves they have little supporting value, they at all times retain contact with and make available the maximum subgrade support. If the sand settles slightly the limerock does also, and if proper side support exists the road should be more resistant than before. Failure

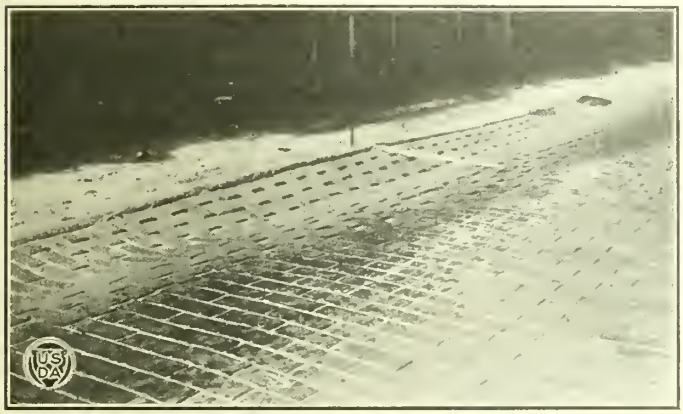


FIG. 21.—Depression in Orlando-Winter Garden road, probably caused by displacement of the sand subgrade, due to vertical movement of the curb.

FIG. 22.—Grooves in Orlando-Winter Garden road which do not impair the smooth riding qualities of the surface.

occurs by excessive settlement, but this can not happen unless the sand base is first displaced. It is possible that if proper side support is supplied a comparatively thin layer of limerock will suffice to prevent loss of sand through the crevices between bricks and to supply additional confinement. Referring to the descriptions of the wider sand-filled roads, it will be recalled that the loose 3-inch brick tops were sufficient to confine the sand subgrade to a remarkable degree, which seems to warrant the above assumption. In the absence of additional side support, a base of considerable thickness will probably be required to carry heavy traffic.

FIG. 23.—Section of a cement-grout-filled surface laid through a swamp section south of Kissimmee which has conformed to changes in the sand subgrade without becoming rough.

CONCRETE BASES VIEWED AS UNNECESSARY IN FLORIDA.

The use of limerock is warranted only because of the high supporting value of the sand subgrade. If the subgrade support were so low as to require a base course having beam resistance equivalent to that furnished by a 6-inch thickness of concrete, it is questioned whether 10 or even 12 inches of limerock would be adequate.

The behavior of the Florida surfaces, however, has not indicated that such a base is required. Notwithstanding considerable opinion to the contrary, there was ample evidence to show that grooving and settlement were not caused by recent traffic conditions. These defects began to form in many cases soon after completion of the road. In 1918 the grooves on the Orlando-Winter Garden Road had so changed the transverse profile of the surface that the rear wheels of a passing road roller had contact only at their inner edges and thus cracked a number of bricks. The bricks in the Jacksonville-St. Augustine Road rattled during the passage of vehicles in 1918 just as they did in 1923. Naturally, where the roads were entirely neglected, deterioration developed rapidly with increase of traffic, but this was a fault of the organization in charge rather than of the surface.

FIG. 24.—Limerock base before rolling.

Although little could be learned of the traffic which the various roads had accommodated, a most important factor noted was that, to facilitate travel through sand and, in some instances, to comply with traffic regulations, *but few trucks were equipped with other than pneumatic tires.* In this discussion of base courses it is assumed that this type and not solid-tired traffic is to be accommodated in the future. The gross load limitation of the State law is 16,000 pounds on pneumatic-tired vehicles and 8,000 pounds on solid-tired vehicles.

THE VOLUME OF TRAFFIC.

As to the number of vehicles carried, it is indicated by information supplied by the Florida State Highway Commission that between January and June, 1923, considering only trucks, the road between Lakeland and Tampa ranked first, with 200 per day; that between Orlando and Kissimmee, second, with 169; and that between Sanford and Orlando, third, with 157 trucks per day. The number of trucks averaged from 10 to 30 per cent of the total number of vehicles. Since the road between Lakeland and Tampa carried traffic in two lines, it is apparent that the narrow road between Sanford and Kissimmee received the greater intensity of traffic since it was carried in one line.

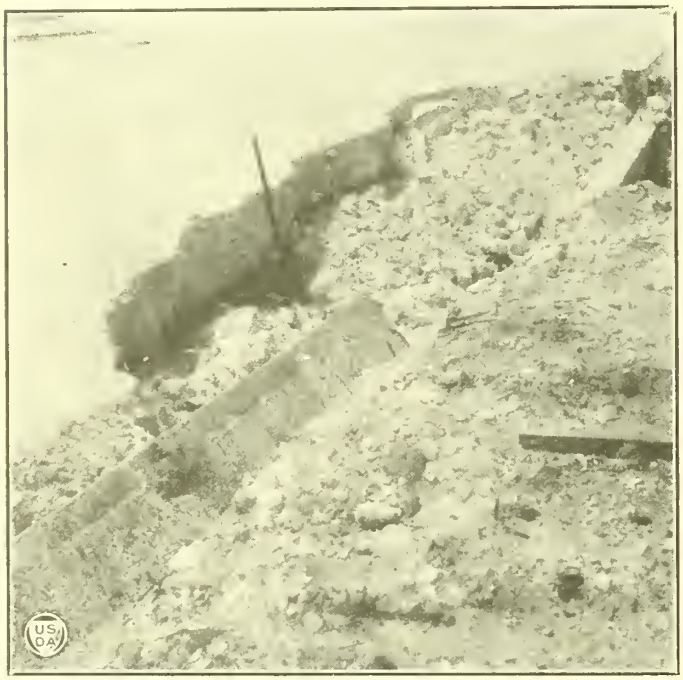


FIG. 25.—Failure of limerock base due to washout of sand subgrade. Note the smooth surface obtained by rolling.

Current opinions in regard to the necessary thickness of limerock bases are reflected by the bases employed in present construction. Many of the counties and towns are laying a 6-inch layer, in some instances covered only by a bituminous surface treatment. The Jacksonville-Atlantic Beach Road is being constructed with 7 inches of loose material, or about 5 inches of compacted base. The present Federal-aid specifications require 8 inches. As previously noted, the writer does not believe that a base of uniform thickness is the most economic preventive of the Florida brick road defects. No matter how thick the base is made, the

FIG. 26.—Limerock road compacted by traffic. Laid in 1914.

sides will be relatively weaker than the middle. With few exceptions the condition of the pavements observed indicated that additional shoulder resistance is most essential. There is reason to believe that with a given quantity of limerock, if adequate shoulders are first laid and a confining base layer is made of the material that is left, a considerably more resistant surface will be secured than if all of the rock is used for a base of uniform thickness. On a road 18 feet wide, for instance, a 4-inch base and shoulders 24 by 8 inches will possibly offer greater resistance than a 6-inch base. By this means effective additional road width is also secured.

In those surfaces along which curbs are not used, increasing the width of base with material secured by reduction of thickness is recommended. A base 21 feet wide by 6 inches thick might be more effective under a road 18 feet wide than a base 18 feet wide and 7 inches thick.

The most economic distribution of limerock between base course and shoulders can be determined only by construction and observation of experimental sections. By this means also can be determined the advisability of using wooden curbs when limerock bases and shoulders are employed.

In connection with the use of the limerocks, it is to be emphasized that they must be thoroughly puddled and compacted in order that danger of uneven settlement in the future may be prevented.

FIG. 27.—Limerock road compacted by road roller. Built in 1913 and carefully maintained, it is now a very satisfactory, hard, and compact road.

ROAD MATERIAL TESTS AND INSPECTION NEWS

WATER REQUIRED FOR STANDARD MORTAR CONSISTENCY DETERMINED BY NEW METHOD.

To determine the amount of water required for standard consistency of mortars made with natural sand is the purpose of one of the investigations now being conducted by the physical laboratory of the United States Bureau of Public Roads.

The greatest problem in connection with the strength-ratio test for concrete sands has been that of properly gauging the amount of water for the natural sand mortar for comparison with standard, Ottawa-sand mortar. Each laboratory acts according to its own judgment and, on account of the large personal equation involved, the result has been a wide variation in test results.

Studies made in the bureau's laboratory have included a number of methods such as computation of surface area, determination of point of maximum density, measurement of consistency by the flow table, and various modifications of each of these methods. Lately the problem has been attacked from a new angle in that an effort is being made to determine experimentally the amount of water required to cover the surfaces of the sand grains and to provide for absorption. Knowing this it is only necessary to add cement and the additional water for normal-consistency cement paste to put the mortar in a standard condition.

Briefly, the method is as follows: A representative sample of the sand passing the one-fourth-inch screen and retained on the 100-mesh sieve is dried, weighed, and saturated with water; it is then transferred to a filtering tube of special design and a suction of 15 inches vacuum applied for 10 minutes, moist air being supplied to the sample during suction. The water retained by the sample is determined by weighing. By comparison of this method with the usual method on a number of samples a constant of 1.6 has been found necessary as a multiplier to give an adequate percentage of water for the sand. All material passing the 100-mesh sieve is considered as of cement size and is treated accordingly. The formula is:

$$N = 1.6rW + n\ (C + w)$$

in which

N = required weight of water for mortar mix in grams.
r = percentage by weight of water retained by test of $\frac{1}{4}$–100 sand.
W = weight in grams of $\frac{1}{4}$–100 sand in mortar mix.
n = normal consistency of cement.
C = weight of cement in mix in grams.
w = weight of sand passing 100-mesh sieve in mortar mix in grams.

This method promises to be more reliable than any heretofore tried out, but the amount of data available is as yet not sufficient to form the basis of definite recommendations.

BETTER CONCRETE ROADS.

The conviction prevails that there is room for marked improvement in the methods employed in the construction of concrete roads and the manipulation of the materials used. This view is supported by the results of compression tests on cores drilled from pavements which indicate a wide variation in strength, the maximum being often from 50 to 100 per cent greater than the minimum. A number of factors are known to contribute to the strength of the concrete, but in most instances the cause of defects can not be attributed to any specific deficiency of construction.

It is the common belief that there is great need for the introduction of a more accurate control of the construction process. Certain principles have been evolved through research which are generally accepted. The present problem is one of practically applying these principles to construction. In an attempt to solve this problem, the United States Bureau of Public Roads has in prospect the construction of an experimental concrete pavement cooperatively with one of the Eastern States.

Assuming that the design and specifications are established and that the materials have been selected for the job, the mechanical factors which are frequently neglected and which have an important influence on the resulting concrete are as follows: The accurate measurement of aggregates which involves the bulking of fine aggregates due to moisture content, the correct grading of the coarse aggregate, a uniformly proper consistency and adequate curing of the pavement. With an accurate control of these factors together with certain refinements of construction, it is expected to secure an improved workability of the mixture, a homogeneous concrete of more uniform strength, a smoother riding surface and perhaps an increased yield for a given mix.

In order to obtain a control of this kind and apply it, a study of road-building equipment is being made by the bureau with a view to selecting or modifying available equipment, or devising equipment particularly suited to the requirements. For a more precise means of measuring aggregates a comparison of the weight and volumetric methods is being made. An investigation of the bulking of sand due to moisture content is being conducted with a modern volumetric plant. With regard to the grading of coarse aggregates, consideration is being given to the idea of delivery to the job in screened sizes and combining them at the central proportioning plant. It is also important that some effective means of controlling the consistency be devised.

By means of this experimental construction which is proposed for the coming season, it is hoped to develop the necessary equipment and a practical control of construction which will produce a pavement of better quality and uniformity without adding greatly, if at all, to the cost.

A NOTE ON THE DISTILLATION TEST FOR ROAD TARS.

Considerable variations in the results obtained from this test by different laboratories are caused by the use in some laboratories of uncorrected thermometers, and in others of thermometers calibrated in the vapors of pure substances with known boiling points. The procedure used by almost all producers and State highway testing laboratories for this test is that adopted by the American Society for Testing Materials, serial designation D20–18, which requires that the thermometer shall be set up in the apparatus as in the test, with water, napthalene and benzophenone as distilling liquids. However, the standard method does not state clearly how the temperatures indicated by these calibrations are to be used, and the result has been that only a minority of the laboratories engaged in testing tars make use of this method of determining corrections to be applied to the thermometer in arriving at the true temperature of the distilling vapors. Other laboratories take actual readings of the thermometer as fractionating points. The corrections by the first method may amount to from 5° to 10° at 300° C.

The difference in percentage of total distillate to 300° C. is not a very serious matter, ranging up to 3 or 4 per cent; but when the effect of this distillate on the softening point of the residue is considered, it is often found to be sufficient to place the material tested either within or without the requirements of the specification, depending upon whether a corrected or uncorrected thermometer is used in the test. A variation of 8° or 10° C. in softening point may readily occur.

There are good arguments in favor of either use of the thermometer. The object here is merely to call attention to the influence a variation in practice may have on test results, and point out that its bearing on the interpretation of distillation results and on the selection of specification requirements should be kept in mind. It is obviously desirable that all producers and consumers of road tar products shall agree upon a uniform procedure.

SOFTENING-POINT TEST.

The ring-and-ball method for conducting the softening-point test on bituminous road materials is used by the majority of State testing laboratories, but some States use the cube method in at least some softening-point determinations.

The use of two different methods makes for confusion and lack of uniformity. The universal use of one method is highly desirable, especially in view of the difference in results obtained by the two methods. The committee on tests and investigations of the American Association of State Highway Officials has recommended the use of the ring-and-ball method (see Department of Agriculture Bulletin No. 949, p. 52) and the Bureau of Public Roads in its Typical Specifications for Bituminous Road Materials (Department of Agriculture Bulletin No. 691) stipulates the use of the ring-and-ball method.

TESTING SIEVES.

An effort is being made by the American Society for Testing Materials working in cooperation with the United States Bureau of Standards, United States Bureau of Public Roads, American Association of State Highway Officials, and the several manufacturers, to adopt a practical standard screen scale for testing sieves. Several meetings have been held and enough progress made to indicate that in all probability the screen scale of the United States Bureau of Standards, somewhat modified as to permissible tolerances in both average opening and wire diameter, will be adopted. The general adoption of such a screen scale by all users and manufacturers of testing sieves will greatly simplify the use of the sieves in the laboratory, because it will be possible in all cases to correlate the sieve number with the size of the actual opening and thus definitely fix the size of material under examination.

Should this screen scale be adopted it has been suggested that the Bureau of Standards be requested to furnish sieve correction factors for sieves other than the standard No. 200, in order that anyone who desires to have a sieve calibrated for special purposes can do so

States.	Fiscal years 1917–1923.			Fiscal year 1924.									Balance of federal aid fund available for new projects.
	Projects completed prior to July 1, 1923.			Projects completed since June 30, 1923.			*Projects under construction.			Projects approved for construction.			
	Total cost.	Federal aid.	Miles.	Total cost.	Federal aid.	Miles.	Estimated cost.	Federal aid allotted.	Miles.	Estimated cost.	Federal aid. allotted.	Miles.	
Alabama	$3,789,011.74	$1,794,703.21	367.9	$286,155.95	$136,517.74	21.6	$14,195,713.05	$7,086,774.06	897.9	$652,035.75	$278,708.11	4.4	$414,207.32
Arizona	6,518,025.52	3,195,689.68	372.1	786,232.63	478,273.20	67.1	2,692,535.69	1,585,993.40	211.0	476,241.20	291,030.97	35.8	891,710.19
Arkansas	8,623,205.72	3,308,572.12	736.2	2,268,532.68	1,020,541.62	186.5	4,311,917.55	1,757,591.09	229.6	3,403,273.67	816,200.00	195.5	900,638.10
California	8,495,547.60	3,538,967.90	340.5	2,964,861.97	1,310,621.71	140.5	11,771,123.33	6,275,620.89	443.6	808,281.23	436,250.89	23.8	3,066,853.83
Colorado	6,470,695.56	3,136,238.67	426.6	964,051.33	531,160.80	50.2	4,252,262.14	2,299,496.02	175.2	272,603.54	152,983.61	12.2	2,078,519.84
Connecticut	2,568,643.95	1,026,148.85	60.7	128,610.43	64,305.21	3.8	1,877,498.46	727,644.54	36.5	1,270,571.00	415,979.70	21.5	671,602.79
Delaware	2,196,610.22	621,154.83	45.2	718,598.97	317,815.00	22.7	988,468.80	389,767.50	27.5	76,668.90	31,961.10	0.2	13,206.40
Florida	69,466.31	29,700.63	15.6	604,477.52	288,175.17	23.3	8,570,568.56	4,228,785.06	255.3	1,037,874.30	440,221.40	13.7	412,668.22
Georgia	13,960,499.47	6,363,703.60	925.5	2,523,160.62	1,252,345.65	255.7	8,227,627.15	3,990,683.96	686.1	1,483,127.83	704,483.48	103.7	155,657.32
Idaho	6,880,673.73	3,272,111.01	426.1	828,865.25	473,209.67	53.0	1,666,084.06	1,049,866.21	115.6	678,769.61	387,316.15	52.3	558,510.95
Illinois	25,211,526.23	11,414,291.96	743.5	339,902.78	166,739.78	13.1	14,623,252.25	7,277,387.20	494.9	1,414,852.39	698,013.63	43.1	676,191.44
Indiana	5,513,487.76	2,668,277.55	157.0	1,926,324.67	919,807.21	64.4	10,899,206.06	5,327,846.39	345.7	3,618,885.20	1,798,033.96	146.4	1,658,523.57
Iowa	16,040,079.62	6,056,136.48	1,059.4	4,779,027.83	2,093,746.23	472.6	7,559,578.44	3,828,518.14	677.6	2,763,109.43	1,257,950.00	101.8	21,537.82
Kansas	12,216,428.90	4,248,438.34	349.7	2,160,141.50	741,151.46	72.0	18,334,748.58	6,530,966.77	604.5	1,294,142.18	577,646.46	50.9	1,119,855.93
Kentucky	6,620,289.80	2,789,991.15	285.0	3,892,469.01	1,683,185.33	142.4	7,514,885.23	3,630,065.39	326.5	471,726.68	235,863.33	19.0	621,049.35
Louisiana	7,131,394.90	3,157,128.03	590.4	1,116,208.52	367,989.39	54.2	5,430,504.00	2,582,338.36	355.9				162,684.63
Maine	4,662,854.19	2,242,963.03	159.8	2,248,204.59	1,056,972.35	70.9	907,304.26	451,804.19	34.8	527,035.66	263,238.99	26.9	388,540.08
Maryland	5,309,394.94	2,524,843.54	185.0	1,427,449.76	676,878.38	57.3	1,601,912.79	669,855.75	52.0				141,427.32
Massachusetts	6,841,975.16	2,802,957.88	163.7	1,045,304.51	424,053.44	20.4	5,223,923.29	1,932,439.52	104.6	1,603,248.87	523,760.02	23.1	1,146,762.92
Michigan	9,528,041.53	4,343,817.14	363.4	2,331,745.70	963,110.18	70.4	10,690,885.25	5,230,876.33	428.7	357,646.00	178,022.50	11.5	2,937,121.12
Minnesota	19,828,671.37	8,004,277.87	1,893.4	4,208,889.87	1,881,565.20	398.6	7,294,164.82	2,953,348.91	512.1	456,531.47	208,500.00	73.6	149,820.46
Mississippi	4,746,771.31	2,284,557.86	462.4	2,379,799.09	1,170,449.79	128.5	8,391,393.48	4,122,708.66	535.6	352,642.76	152,986.68	27.8	506,198.36
Missouri	5,272,757.08	2,338,848.60	365.6	5,120,381.37	2,452,903.48	342.4	14,599,273.33	6,998,185.49	655.0	1,430,832.46	715,416.14	87.0	2,991,348.54
Montana	7,935,185.30	3,896,053.50	711.5	789,664.09	412,808.75	62.0	1,847,534.34	1,295,755.10	194.1	1,123,265.98	589,381.89	124.4	3,227,934.57
Nebraska	3,159,814.47	1,449,675.04	475.6	4,103,713.46	1,962,846.66	821.7	5,393,756.91	2,653,701.57	696.3	1,198,100.02	599,049.93	132.0	3,208,517.46
Nevada	3,049,109.29	1,562,111.35	203.6	486,806.51	356,779.95	33.3	3,563,552.53	3,072,334.51	308.4	400,982.30	301,806.36	10.4	649,665.58
New Hampshire	2,336,997.00	1,129,004.69	140.5	728,350.97	353,161.78	30.5	603,215.61	285,719.58	21.8	238,984.11	110,652.73	6.8	190,800.21
New Jersey	5,423,362.96	1,975,783.52	112.1	1,794,735.56	562,527.10	29.4	8,552,430.81	2,153,320.04	49.0	823,264.45	174,640.00	10.9	786,563.31
New Mexico	4,706,909.12	2,433,663.68	617.1	599,377.33	325,186.00	97.2	4,061,029.74	2,519,288.58	544.5	1,553,929.30	1,019,519.32	91.2	1,106,145.54
New York	9,826,633.82	4,358,659.70	290.6	4,552,518.80	1,972,774.97	151.1	22,208,223.68	8,783,006.83	531.9	8,690,700.00	3,241,612.50	185.2	4,688,988.14
North Carolina	11,770,363.76	5,325,763.37	826.8	797,369.21	350,994.29	57.9	8,713,745.41	3,081,726.31	233.2	2,508,178.50	1,147,748.72	68.1	690,772.15
North Dakota	5,142,157.55	2,492,599.09	792.5	3,261,595.19	1,614,534.18	699.2	2,456,478.94	1,183,150.78	384.7	205,979.82	102,989.89	39.4	1,791,673.93
Ohio	22,840,050.09	7,966,325.17	662.8	9,618,128.44	3,686,542.82	281.0	11,574,250.14	4,384,022.84	310.2	1,797,402.18	591,426.05	60.5	716,042.43
Oklahoma	9,795,634.54	4,417,788.46	371.5	3,041,827.34	1,396,486.82	121.3	8,043,124.57	3,860,977.42	354.9	1,138,481.23	549,825.45	56.7	558,435.14
Oregon	10,391,362.48	4,782,736.38	534.1	1,368,500.56	849,646.22	104.9	2,963,118.99	1,600,164.90	160.6	138,208.95	65,675.96	17.5	31,105.39
Pennsylvania	33,090,029.45	12,615,058.79	652.8	2,578,716.07	1,054,171.61	54.6	7,948,607.18	2,699,782.50	140.0	5,899,493.19	1,622,725.00	100.3	3,243,921.89
Rhode Island	1,484,075.35	647,634.96	38.6	140,504.75	61,669.36	3.5	1,227,634.35	481,683.13	26.7				376,428.68
South Carolina	6,181,485.41	2,885,754.37	655.1	1,790,048.23	721,199.07	235.3	6,460,068.45	2,595,122.83	471.8	694,554.49	219,707.61	68.0	211,733.95
South Dakota	4,758,907.24	2,320,007.33	534.5	3,307,648.50	1,634,101.11	393.4	5,404,391.94	2,772,852.75	669.5	671,630.40	309,731.78	61.7	472,842.85
Tennessee	3,210,970.94	1,535,762.18	112.5	1,317,446.92	647,258.24	62.5	14,596,401.31	7,109,466.98	472.7	1,775,789.62	654,004.29	48.2	449,404.34
Texas	29,172,501.06	11,094,069.77	2,259.6	11,130,736.18	4,209,946.93	754.1	24,222,455.38	9,701,177.66	1,495.8	4,036,658.84	1,602,881.78	277.0	705,967.10
Utah	2,175,007.41	1,196,437.68	116.6	804,952.96	461,959.20	76.7	3,631,445.27	2,272,865.38	249.4	716,980.03	531,949.40	39.1	805,519.44
Vermont	1,370,715.63	669,433.75	53.1	198,959.70	99,240.82	6.8	1,474,576.24	721,509.24	47.6	290,856.75	128,848.50	6.5	549,321.42
Virginia	6,823,978.36	3,279,029.56	402.5	2,081,874.91	968,191.98	94.7	9,009,188.44	4,247,927.51	334.7	911,299.18	270,025.88	45.5	379,215.52
Washington	10,356,980.82	4,819,105.70	419.8	618,239.13	273,551.37	35.4	2,354,341.13	1,110,300.00	97.0	398,953.23	194,500.00	14.7	375,912.76
West Virginia	4,100,225.73	1,800,932.41	206.3	1,276,510.70	511,859.12	42.2	4,844,949.70	2,136,366.50	163.9	485,387.10	216,228.55	19.0	290,469.95
Wisconsin	15,504,691.39	5,996,989.24	1,077.6	1,923,159.81	856,321.21	153.1	3,970,457.96	1,857,167.49	206.3	109,324.13	54,662.06	9.3	3,035,710.68
Wyoming	4,601,439.36	2,210,774.20	534.8	990,072.46	582,755.04	102.7	4,028,357.61	2,484,621.96	281.2	243,934.58	56,705.52	35.5	416,122.15
Total	407,704,641.14	174,044,673.82	23,297.2	104,345,854.32	46,398,032.59	7,236.1	340,778,161.20	155,992,576.22	16,652.4	60,502,438.51	24,920,866.29	2,602.1	50,643,851.08

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost, $62,976,042.25; federal aid, $27,799,234.86; miles, 2,885.2.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS.

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets, nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

REPORTS.

Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.
Report of the Chief of the Bureau of Public Roads for 1920.
Report of the Chief of the Bureau of Public Roads for 1921.
*Report of the Chief of the Bureau of Public Roads for 1922. 5c.
*Report of the Chief of the Bureau of Public Roads for 1923. 5c.

DEPARTMENT BULLETINS.

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*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
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*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.

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*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
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161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.
Public Roads Vol. III, No. 25. Automobile Registrations, Licenses, and Revenues in the United States, 1919.
Vol. III, No. 29. State Highway mileage, 1919.
Vol. III, No. 36. Automobile Registrations, Licenses, and Revenues in the United States, 1920.
Vol. IV, No. 5. Automobile Registrations, January 1 to July 1, 1921.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH.

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

FUNDAMENTAL PRINCIPLES OF HIGHWAY FINANCE

The wide variation in the present status of highway development in the several States prevents the adoption of a uniform policy for securing the funds necessary to the annual highway budget and expending these funds. Generally speaking, however, these principles may be enunciated.

(*a*) States in the initial stage of highway development should issue bonds to defer that portion of the annual charge for construction which would overburden either property or the road user.

(*b*) States where original construction programs are well under way can, in the main, finance further expenditure for construction by bond issues devoted to deferring the cost of special projects.

(*c*) States where original construction is practically completed are concerned chiefly with maintenance and reconstruction and should depend on current funds, save in cases of emergency.

(*d*) The maintenance of interstate and State roads should be a charge against the road user.

(*e*) Roads serving a purely local purpose will generally require only light upkeep, and this should properly be a charge against the adjacent property, which in this case is the first and often the only beneficiary.

(*f*) No road should ever be improved to an extent in excess of its earning capacity. The return to the public in the form of economic transportation is the sole measure of the worth of such improvements.

5:3

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 3 ▽ MAY, 1924

GENERAL VIEW OF EXPERIMENTAL PAVEMENTS OF THE BUREAU OF PUBLIC ROADS AT ARLINGTON, VA.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

VOL. 5, NO. 3 | MAY, 1924 | H. S. FAIRBANK, Editor

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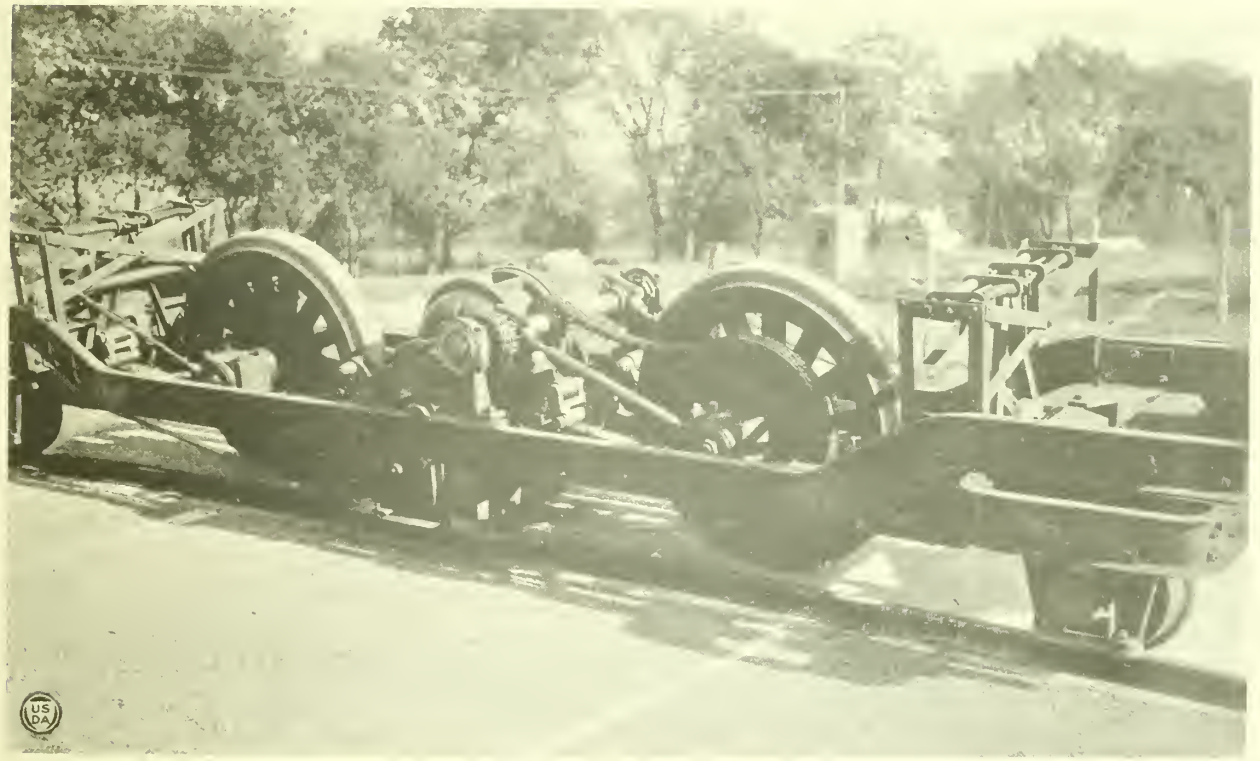
WEAR OF CONCRETE PAVEMENTS TESTED

REPORT OF ACCELERATED TESTS BY U. S. BUREAU OF PUBLIC ROADS

By F. H. JACKSON, Senior Assistant Testing Engineer, and J. T. PAULS, Highway Engineer, U. S. Bureau of Public Roads



The loaded, rubber-tired wheels of the testing machine bear with their full weight on the concrete. The machine is guided by the rails at the side of the experimental sections

THE accelerated wear tests of concrete pavements conducted by the Bureau of Public Roads, at Arlington, Va., during the past 18 months, have reached the stage where it is possible to draw fairly definite conclusions from the data so far obtained. The 62 concrete test sections, each 4 feet wide by about 10 feet long, and constructed in the form of a circle approximately 625 feet in circumference, have been subjected to approximately 300,000 passages of a solid-rubber-tired truck wheel loaded to 3,000 pounds and traveling around the circle over the same path at a speed of 22 miles per hour. Careful observations of the lateral distribution of traffic across actual pavements indicate that about 10 per cent of the total passes over a 6-inch width at the point of greatest concentration. This is based on traffic passing in two directions over an 18-foot road. On this basis it is estimated that the experimental pavement sections have been subjected to a traffic equivalent to about 1,500,000 two-ton trucks operating at a speed of 22 miles per hour assuming the distribution indicated by the above observations. In addition, the test sections have been subjected to about 50,000 passages of a rubber-tired wheel loaded with the same weight and equipped with nonskid chains and the same number of passages of a plain rubber-tired wheel. The wheels in this part of the test traveled over a new path. It will be seen, therefore, that the equivalent of an enormous volume of traffic has been put upon the pavement—an amount probably equivalent to several years traffic on the average concrete road.

It was, of course, impossible to reproduce on the test sections in the short space of a year and a half the effect of freezing and thawing which, through a period of years, may weaken actual roads so that they become less resistant to traffic. It was likewise impossible to reproduce the effect of differential expansion and contraction of the thin, rich mortar top and the mass of the concrete, which, caused by wide variations in temperature, may also, in time, lower the resistance of a pavement. The essential time element is lacking. In these respects, therefore, the test did not duplicate actual conditions, and for this reason the differences in the behavior of the several sections under examination may not be so marked as they normally would have been.

THE PURPOSE AND RESULTS OF THE TEST

The primary object of the test was to determine what relations, if any, exist between the surface behavior of concrete roads under traffic and the

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various laboratory tests for quality of the aggregates. Incidentally it was also deemed desirable to determine the relation between the wear as produced by traffic, as well as by laboratory test, and the various physical properties of the concrete, such as crushing strength, transverse strength, modulus of elasticity and absorption. All of these tests were made on the concrete used in each of the 62 test sections.

Specifications for aggregates for concrete pavements are usually drawn to cover the gradation and the quality of the material furnished. The gradation, or proportion of the various sizes, especially as regards aggregates which are crushed and screened, depends largely on factors which can be controlled, so that the engineer may obtain almost any desired gradation of sizes provided he is willing to insist upon it. The quality of the material, however, can not be fixed in so arbitrary a fashion. Due to the high cost of transportation, the aggregates for any given improvement

Placing the concrete

must generally be found within a relatively short distance of the site of the work. Therefore, as the quality and character of materials varies so greatly in different parts of the country, it follows that many different kinds and types of aggregates must be employed in the construction of concrete roads. All that can be done is to draw specifications in such a way as to insure the use of the best material available for the work. In so doing, however, there is always a minimum limit beyond which it is unsafe to go.

What are the safe test limits for the various kinds of aggregates used in concrete road construction? Do the present limits for wear, strength, etc., insure the selection of safe materials, or, on the other hand, do they unduly add to the cost of construction by setting laboratory requirements too high? These are some of the questions which the accelerated wear tests were designed to answer.

The detailed conclusions which it has been possible to draw from the test are set forth fully at the end of this article. Outstanding from these conclusions are two facts of major importance. These are:

1. That rubber-tired traffic alone does not appreciably abrade the surface of a concrete pavement.

2. That there is no consistent relation between the results of the "tensile-strength-ratio" test for sand, the compression or transverse strength tests or the Talbot-Jones wear test of concrete, and the wear-resisting properties of concrete pavements.

DESCRIPTION OF THE TEST SECTIONS

The test sections were constructed in the form of a circular concrete pavement approximately 625 feet in circumference and 4 feet wide. There were 62 sections, each about 10 feet long. A general view of the experiment, showing also the stability experiments on bituminous surfaces, is given in the cover illustration. Inasmuch as the only object of this test was to bring out differences in the amount and character of wear or surface disintegration due to traffic, the sections were constructed heavily enough to prevent the possibility of any structural failure. The test sections were laid in two courses, the lower 8 inches, laid directly on the subgrade, consisting of 1:1½:3 concrete, using Potomac River sand and gravel as aggregates. A high-grade Portland cement passing all of the American Society for Testing Materials requirements was used throughout the experiment. On one side of the circle it was found necessary to construct the test pavement on a fill. The base over this section was further strengthened by the use of a reinforced concrete T-beam section constructed with the beams running longitudinally. This design proved entirely adequate and there has been no evidence of failure due to settlement on any test section. The test concrete forming each section was cast directly upon the green concrete base and was 4 inches in depth. The total depth of pavement therefore was in every case at least 12 inches of monolithic concrete. No expansion joints were provided, the concrete in each day's run being finished to a vertical wooden bulkhead which was removed the next day and the fresh concrete deposited directly against that placed the day before. As a result of this procedure construction joint cracks have formed between several of the sections. Additional contraction cracks have also developed in a few places. None of these cracks, however, has affected the stability of the pavement in the slightest degree.

The test aggregates were proportioned by loose volume, each batch consisting of one bag of cement, 1½ cubic feet of sand, and 3 cubic feet of coarse aggregate, except on those sections where other proportions were tried for experimental purposes. The concrete was mixed in a one-bag, power-driven, batch mixer; and every effort was made to have the consistency as nearly uniform as possible except in the group where it was varied designedly. In view of the widely varying characteristics of the materials it was found impossible to control this factor absolutely. The consistency of the concrete going into each section was measured by means of the slump test and flow table. The average results of the consistency tests are given for each section in Table 1. The concrete in practically all cases was of medium consistency and rather inclined to be too dry than too wet. All the coarse aggregates, unless otherwise noted were graded from 1½ inches uniformly down to one-fourth inch in size. Potomac River concrete sand, used as the standard fine aggregate, was well graded and of known quality. The concrete in each section was struck off by means of a wooden strike board and was finished with a wooden hand float.

As the testing machine was expected to operate at a speed of approximately 20 miles per hour around a circle with a 100-foot radius, a superelevation of about 7 inches was found necessary. This was not as much as was theoretically desirable, but it was found impracticable to properly finish the concrete surface on any greater slope. Iron bolts were cast in the pavement at regular intervals for the purpose of holding the rails which were to guide the testing machine. The rails were laid 42 inches center to center. Various details of the construction of the test sections are shown in the accompanying illustrations.

TABLE 1.—*Consistency of concrete used in the test sections, showing relation between slump and flow*

Section No.	Average slump	Average flow	Section No.	Average slump	Average flow
	Inches			*Inches*	
1	3	165	32	1	135
2	3	165	33	4	180
3	2	155	34	1	140
4	3	160	35	2	175
5	3	165	36	4	170
6	2	140	37	3	150
7	2	160	38	2	150
8	1	135	39	2	150
9	2	150	40	3	160
10	1	140	41	2	145
11	3	170	42	2	150
12	3	170	43	4	150
13	2	145	44	1	130
14	1	140	45	4	175
15	2	155	46	-	-
16	2	155	47	2	155
17	2	155	48	1	120
18	2	150	49	1	135
19	2	150	50	2	155
20	3	170	51	1	135
21	2	150	52	2	150
22	2	160	53	4	165
23	2	150	54	2	155
24	2	155	55	.....	145
25	0	120	56	2	165
26	1	145	57	1	145
27	1	120	58	1	130
28	3	160	59	3	150
29	5	180	60	3	155
30	2	150	61	1	135
31	3	165	62	3	150

Each result is the average of 4 tests. The slump test was made with the 4 by 8 by 12-inch truncated cone. The flow test was made with the 30-inch flow table, using 15 drops from a height of one-half inch. The flow is the final diameter of a truncated cone whose original diameter was 100.

SECTIONS ARRANGED IN FIVE GROUPS FOR STUDY OF VARIOUS CHARACTERISTICS

The sections were divided for purposes of study into five major groups, as shown in Table 2. The source, character, and physical characteristics of the various coarse aggregates are given in Table 3 and the corresponding information for the fine aggregate in Table 4. The sections in Group I were constructed with the purpose of studying the effect of variations in quality of crushed stone as coarse aggregate. Ten samples of stone varying in quality from hard, tough traps to very soft limestones were used for this purpose. In Group II the effect of variations in the quality of gravel as a coarse aggregate was investigated, and for this purpose nine samples of gravel from various parts of the country were obtained. Group III comprised eight sections and was for the purpose of studying the behavior of slag as concrete road aggregate. Both the size and quality of the slag as well as the effect of slag sand used as fine aggregate were studied. Four sections were constructed with blast furnace slag, three of them using slag weighing approximately 75 pounds per cubic foot and one in which the slag weighed 56 pounds per cubic foot. Copper and lead smelter slags were used in the other three sections in this group. Group IV was composed of seven sections in which the type of sand was the object of study. The sands used ranged from a very coarse, glacial sand from Wisconsin to a fine, pure quartz sand from South Carolina. Strength ratios varied from 126 to 70 per cent and the fineness modulii from 2.2 to 3.5. The sections in Group V were constructed with the idea of noting the effect of various miscellaneous factors on the surface wear, such as consistency of the concrete, the time of mixing, proportions or cement content, grading of aggregates, and the effect of hydrated lime as an admixture. In addition, sections were constructed using mine chats from Joplin, Mo., and burnt clay as coarse aggregates and limestone screenings in place of sand as fine aggregates. Finally, two sections were built without coarse aggregate, one in the proportion of 1:1½ and the other a 1:3 mortar.

Finishing the concrete

Experimental sections covered with earth for curing

TABLE 2.—*Layout of test sections*

GROUP I

[Sections 1 to 10, inclusive]

Factor studied.—Effect of quality of rock as coarse aggregate.
Fine aggregate.—Potomac River sand.
Coarse aggregate.—Various. (See Table 3.)
Proportions.—1:1½:3 by volume.

GROUP II

[Sections 11 to 19, inclusive]

Factor studied.—Effect of quality of gravel as coarse aggregate.
Fine aggregate.—Potomac River sand.
Coarse aggregate.—Various. (See Table 3.)
Proportions.—1:1½:3 by volume.

GROUP III

[Sections 35 to 42, inclusive]

Factors studied.—Effect of grading and quality of slag as fine and coarse aggregate.
Fine aggregate.—Potomac River and slag sand. (See Table 4.)
Coarse aggregate.—Various. (See Table 3.)
Proportions.—1 : 1½ : 3 by volume.

GROUP IV

[Sections 20 to 26, inclusive and Section 50]

Factors studied.—Effect of grading and quality of sand as fine aggregate.
Fine aggregate.—Various. (See Table 4.)
Coarse aggregate.—Dolomite. (See Table 3.)
Proportions.—1:1½:3 by volume.

GROUP V*a*

[Sections 27 to 34, inclusive and Section 46]

Factors studied.—Effect of consistency, time of mixing and cement content.
Fine aggregate.—Potomac River sand.
Coarse aggregate—
Sections 27 to 34: Dolomite.
Section 46: Potomac River gravel.
Section 27: Very dry consistency.
Section 28: Medium consistency.
Section 29: Wet consistency.
Section 30: Time of mixing, 0 minute.
Section 31: Time of mixing, 2 minutes.
Section 32: Proportions 1:2:4 by volume.
Section 33: Proportions 1:2:3 by volume.
Section 34: Proportions 1:2½:5 by volume.
Section 46: Proportions 1:2½:5 by volume.

GROUP V*b*

[Sections 47 to 54, inclusive]

Factors studied.—Effect of tolerance material in aggregates and one-size stone.
Fine aggregate.—Potomac River sand. (See Table 4.)
Coarse aggregate.—Dolomite graded 1½ to one-fourth inch unless otherwise noted.
Proportions.
Sections 47 to 53: 1:1½:3.
Section 54: 1:1.7:3.2.
Section 47: Fine aggregate contains 15 per cent one-fourth to one-half inch gravel.
Section 48: Coarse aggregate contains 15 per cent stone screenings.
Section 49: Fine aggregate contains 15 per cent one-fourth to one-half inch gravel.
Coarse aggregate contains 15 per cent stone screenings.
Section 50: Fine aggregate, fine Potomac River sand. (See Table 4.)
Section 51: Coarse aggregate, one-fourth to three-fourths inch only.
Section 52: Coarse aggregate, three-fourths to 1½ inch only.
Section 53: Fine aggregate, fine Potomac River sand.
Coarse aggregate, three-fourths to 1½ inch only.
Section 54: Aggregates as in Section 53.
Proportions: 1:1.7:3.2.

GROUP V*c*

[Sections 56 to 62, inclusive]

Factors studied.—Effect of mortar top, hydrated lime and stone screenings.
Fine aggregate.—(See Table 4.)
Coarse aggregate.—(See Table 3.)
Proportions.—1:1½:3 unless otherwise noted.
Section 56: Mortar top 1:1½ by volume.
Section 57: Mortar top 1:3 by volume.
Section 58: 12 per cent hydrated lime by volume added, based on weight of cement.
Section 59: 12 per cent hydrated lime by volume added, based on weight of cement.
Section 60: 12 per cent hydrated lime by volume added, based on weight of cement.
Section 61: Limestone screenings free from dust used in place of sand.
Section 62: Limestone screenings containing dust of fracture used in place of sand.

GROUP V*d*

[Sections 43 to 45, inclusive]

Factors studied.—Effect of mine chats and burnt clay.
Fine aggregate.—(See Table 4.)
Coarse aggregate.—(See Table 3.)
Proportions.—
Section 43: 1:1½:3.
Section 44: 1:1:3.
Section 45: 1:2:2.

TABLE 3.—*Tests of coarse aggregates*

Section No.	Type of material and source	Wear	Hardness	Toughness	Weight per cubic foot		Absorption	Grading
					Solid	Crushed		
		P. ct.			*Lbs.*	*Lbs.*	*P. ct.*	*In.*
1	Argillaceous dolomite: Berkeley County, W. Va.	4.0	17.0	9	176		0.15	¼–1½
2	Altered diabase: Somerset County, N. J.	2.3	18.0	31	183		0.03	¼–1½
3	Quartzite: Minnehaha County, S. D.	1.8	18.7	22	165		0.14	¼–1½
4	Granite porphyry: Llano County, Tex.	2.4	18.7	20	164		0.15	¼–1½
5	Calcareous slate: Texas	4.0	18.3	22	163		0.51	¼–1½
6	Sandstone: Pine County, Minn.	13.8	14.8	4	156		1.23	¼–1½
7	Sandstone: Wise County, Va.	5.3	16.3	11	156		1.55	¼–1½
8	Argillaceous limestone: Huron County, Ohio	6.3	10.3	5	157		3.70	¼–1½
9	Limestone: Presque Isle County, Mich.	9.5	13.3	5	162		0.80	¼–1½
10	Dolomite: Jones County, Iowa	14.5	0.0	4	132		8.96	¼–1½
11	Gravel consisting of rounded fragments of quartz, schist, and sandstone: Potomac River	[1]11.0			158		1.09	¼–1
12	Gravel consisting of rounded fragments of dolomite: Will County, Ill.	[1]9.3			161		1.63	¼–1½
13	Gravel consisting of rounded fragments of limestone, dolomite, chert and sandstone: Warren County, Ind.	[1]9.4			159		1.76	¼–1½
14	Gravel consisting of rounded fragments of quartz, sandstone, limestone and granite: St. Joseph County, Ind.	[2]			160		1.48	¼–1½
15	Gravel consisting of rounded fragments of quartz, rhyolite, andesite and sandstone: Marathon County, Wis.	[2]			164		1.49	¼–1½
16	Gravel consisting of rounded fragments of granite, quartz and diorite: Hampden County, Mass.	[1]14.3			149		0.48	¼–1½
17	Gravel consisting of rounded fragments of quartz: Richland County, S. C.	[1]29.0			144		0.33	¼–1½
18	Gravel consisting of rounded fragments of quartz, granite, sandstone, quartzite and gabbro: Vanderburg County, Ind.	[2]			150		2.11	¼–1

[1] Test made in accordance with Method No. 2, Abrasion test for gravel, U. S. Dept. of Agric. Bulletin 949, p. 3.
[2] Pieces not large enough for this test.

TABLE 3.—*Tests of coarse aggregates*—Continued

Section No.	Type of material and source	Wear	Hardness	Toughness	Weight per cubic foot		Absorption	Grading
					Solid	Crushed		
19	Gravel consisting of rounded fragments of granite, sandstone shale and limestone: Hancock County, Iowa	(2)			149		4.45	¼–1½
20–34	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176		0.15	
35	Blast furnace slag: Montgomery County, Pa	15.9				75		¼–2½
36	Blast furnace slag: Montgomery County, Pa	15.9				75		¼–1½
37	Blast furnace slag: Montgomery County, Pa	15.9				75		¼–1½
38	Blast furnace slag: Montgomery County, Pa	15.9				75		¼–1½
39	Blast furnace slag: Morris County, N. J					56		¼–1½
40	Copper slag: Polk County, Tenn	4.1	18.0	14	220		0.58	¼–1½
41	Copper slag: Middlesex County, N. J							¼–1½
42	Lead slag: Middlesex County, N. J							¼–1½
43	Chert chips: Jasper County, Mo	(2)						[3] 10–½
44	Chert chips: Jasper County, Mo	(2)						[3] 10–½
45	Burnt clay: St. Louis County, Mo	(2)						¼–¾
46	Gravel consisting of rounded fragments of quartz, schist and sandstone: Potomac River	[1] 11.0			158		1.09	¼–1½
47–55	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176		0.15	¼–1½
58	Altered diabase: Polk County, Wis	(2)	18.3	32	185		0.01	¼–1½
59–62	Argillaceous dolomite: Berkeley County, W. Va	4.0	17.0	9	176		0.15	¼–1½

[1] Test made in accordance with Method No. 2, abrasion test for gravel, U. S. Dept. of Agr. Bulletin 949, p. 3.
[2] Pieces not large enough for this test.
[3] Sieve No. 10.

TABLE 4.—*Tests of fine aggregate*

Section No.	Type of material and source	Mechanical analysis. Total per cent passing designated sieves							Silt	Color plate No.	Tensile strength ratio	
		¼ inch	10	20	30	50	100	200			7 days	28 days
									P. c.		P. c.	P. c.
1–20	River sand: Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
21	Bank sand: Merrimack County, N. H	94	66	35	15	3	1	1	0.5	1	71	70
22	Fayette County, Pa	100	93	75	54	21	6	2	1.5	3–4	98	109
23	Bank sand: Marathon County, Wis	99	86	66	34	7	1	1	0.4	1	127	123
24	River sand: Charleston County, S. C	100	98	83	53	20	3	1		1	92	92
25	St. Clair County, Mich	100	68	55	47	18	2	1	0.5	1	145	126
26	Colorado River	100	89	55	16	2	1	1	0.4	1	126	108
27–37	Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
38	Blast furnace slag sand: Montgomery County, Pa	94	51	28	19	12	9	7	5.0		55	94
39	River sand: Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
40	Copper slag sand: Polk County, Tenn	99	85	47	17	6	3	2	1.0		96	132
41–42	River sand: Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
43	"Rolled chat sand": Jasper County, Mo	100	58	33	22	15	11	9	5.4		200	199
44	"Jig Sand": Jasper County, Mo	100	100	94	60	28	12	5	1.4		82	95
45	Burnt Clay: St. Louis County, Mo	99	76	51	33	19	12	8	3.8		108	127
46	River sand: Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
47	Do	83	63	51	39	19	8	4	3.0	1		
48	Do	98	74	60	46	22	9	5	3.0	1	125	114
49	Do	83	63	51	39	19	8	4	3.0	1		
50	Do	100	99	91	65	19	5	2	1.2	1	90	84
51–52	Do	98	74	60	46	22	9	5	3.0	1	125	114
53–54	Do	100	99	91	65	19	5	2	1.2	1	90	84
55	Limestone screenings: Berkeley County, W. Va	95	50	5	0	0	0	0				
56–59	River sand: Potomac River	98	74	60	46	22	9	5	3.0	1	125	114
60	Do	100	99	91	65	19	5	2	1.2	1	90	84
61	Limestone screenings: Berkeley County, W. Va	95	58	30	16	8	4	2				
62	Do	94	69	47	35	28	22	18			202	162

DESCRIPTION OF THE TESTING MACHINE

A view of the testing machine is shown on page 1. It was so constructed as to have approximately the same effect as ordinary truck traffic. Since the width of the test section was only 4 feet the design of the machine included only one front and one rear truck wheel. The solid-rubber-tired wheels were mounted in a frame between two truck springs which served to carry the necessary loading of 600 pounds per inch of tire width on each of the truck wheels, the total load on each wheel being 3,000 pounds. The rear wheel was driven by means of an electric motor. The frame which carried the truck wheels and their loading was mounted in a flexible manner within another frame. The outer frame, equipped with four flanged car wheels, was supported and guided by railroad rails laid at each side of the test sections, and, in turn, guided the inner frame and truck wheels. The whole apparatus was propelled by the truck wheels, which acted freely with their whole weight on the concrete surface. Two complete machines were constructed and coupled together in the test, with each of the truck wheels so aligned that all followed in the same path, approximately 6 inches in width. The abrasive action, thus concentrated upon a narrow path, was greatly accelerated. The machines were operated and controlled through a third-rail system and were run at a speed of from 20 to 22 miles per hour.

TESTS OF CONTROL SPECIMENS

For each section of the pavement, control specimens were made for the purpose of determining the strength of the concrete in compression, cross bending, and resistance to wear. This group of specimens consisted of three 6 by 12 inch cylinders, two 6 by 8 by 48 inch beams, and three 8 by 8 by 5 inch Talbot-Jones wear blocks. A small amount of concrete was taken from each batch for the purpose of fabricating the control specimens, so that the concrete in these specimens may be considered to be fully representative of that in the test sections proper.

Twenty-four hours after molding, the forms were removed and the specimens placed on the section which they represented. An earth covering was then placed over the entire section and kept continuously wet for 14 days, after which the concrete was exposed to the weather. All of the specimens were in an air-dry condition when tested at the age of 90 days.

Compression tests were made in a 200,000-pound Universal testing machine, following in detail the standard American Society for Testing Materials procedure. Deformation readings for determining the modulus of elasticity in compression were taken on two cylinders from each section by means of an improved form of compressometer. This compressometer was of the two-ring, two-dial type and proved very sensitive, movements of the dials taking place under initial load-

ings of 40 pounds per square inch. It is shown mounted on a specimen in the illustration on page 7. The results of the compression tests, including modulus of elasticity in compression, are given in Table 5.

The transverse tests were made on 6 by 8 by 48 inch beams, using a span of 42 inches with center loading. The beams were supported at the ends by specially designed rockers having uniform bearing against the bottom surfaces of the beams and a rocker adjustment upon the supporting plates. These plates were in turn fastened to the flanges of the I-beams at such distance that the centers of the supporting rockers were 42 inches apart, center to center. The deflection-measuring device consisted essentially of three U-shaped brackets of strap iron fastened at the neutral axis on each side of the beam with pointed set screws. Two of the brackets were placed directly over the end supports and the third fastened at the center of the span. This center bracket supported two 0.001-inch Ames dials, one on each side of the beam. Supported horizontally by the set screws of the end brackets were two wooden bars, one on each side of the beam, against which the plungers of the Ames dials rested. The illustration on page 8 shows the details of this deflection measuring apparatus. The loads were applied through a three-fourths inch ball resting on a one-fourth inch steel bar about 7 inches long. They were applied very slowly in increments of 500 pounds until failure occurred. Between the 2,500-pound load and failure the dials were watched very closely to catch the maximum deflection at the point of rupture. Table 6 gives the results of the tests for modulus of rupture and modulus of elasticity in bending for the beams of the 62 test sections.

TABLE 5.—*Compression tests of concrete; age, 90 days*

Section No.	Crushing strength	Modulus of elasticity	Weight per cubic foot	Section No.	Crushing strength	Modulus of elasticity	Weight per cubic foot
	Pounds per square inch	*Pounds per square inch*	*Pounds*		*Pounds per square inch*	*Pounds per square inch*	*Pounds*
1	4,668	5,100,000	154	32	3,380	6,100,000	158
2	3,932	5,000,000	165	33	3,792	5,100,000	155
3	4,139	4,900,000	156	34	3,387	4,700,000	158
4	5,510	4,800,000	159	35	4,827	4,450,000	152
5	4,930	4,800,000	156	36	4,987	5,000,000	149
6	6,143	3,550,000	148	37	4,772	4,600,000	149
7	5,015	1,650,000	149	38	4,827	3,950,000	142
8	4,786	4,000,000	153	39	4,333	4,350,000	135
9	4,883	4,100,000	155	40	6,697	6,200,000	193
10	4,878	3,450,000	151	41	6,003	6,300,000	187
11	4,163	3,300,000	155	42	6,328	3,450,000	180
12	4,345	4,250,000	154	43	4,170	4,000,000	143
13	4,770	4,300,000	155	44	3,735	4,300,000	140
14	4,192	4,000,000	156	45	5,165	2,300,000	116
15	5,203	4,400,000	159	47	4,290	5,000,000	156
16	4,883	3,200,000	180	48	5,363	5,100,000	159
17	3,827	2,750,000	155	49	4,597	4,950,000	159
18	4,537	5,100,000	152	50	4,988	5,250,000	158
19	3,480	3,350,000	152	51	4,852	5,150,000	162
20	3,902	5,150,000	158	52	5,263	6,150,000	163
21	4,240	5,250,000	161	53	5,372	5,300,000	160
22	3,615	4,800,000	156	54	5,100	4,670,000	156
23	4,923	7,000,000	159	55	1,741	3,100,000	135
24	4,925	5,500,000	160	56	7,083	3,800,000	143
25	5,197	5,700,000	163	57	3,887	3,100,000	140
26	5,585	6,200,000	160	58	5,533	6,200,000	165
27	4,530	6,250,000	158	59	4,205	5,300,000	160
28	4,648	5,550,000	158	60	4,630	5,430,000	154
29	4,268	5,100,000	158	61	3,635	4,550,000	152
30	3,842	4,800,000	161	62	3,675	4,500,000	158
31	3,992	5,100,000	159				

TABLE 6.—*Transverse tests of concrete; age, 90 days*

Section No.	Modulus of rupture	Modulus of elasticity	Section No.	Modulus of rupture	Modulus of elasticity
	Lbs. sq. in.	*Lbs. sq. in.*		*Lbs. sq. in.*	*Lbs. sq. in.*
1	480	4,735,000	32	592	4,225,000
2	512	4,050,000	33	728	4,395,000
3	677	3,265,000	34	545	5,030,000
4	607	3,845,000	35	589	4,390,000
5	483	4,000,000	36	690	4,545,000
6	632	3,335,000	37	597	4,030,000
7	499	1,992,500	38	774	3,600,000
8	692	3,550,000	39	729	3,525,000
9	509	3,860,000	40	754	5,330,000
10	599	3,665,000	41	573	4,365,000
11	527	3,265,000	42	558	3,255,000
12	659	4,410,000	43	627	4,005,000
13	517	4,525,000	44	692	3,985,000
14	538	3,590,000	45	416	2,045,000
15	536	4,095,000	47	623	4,230,000
16	532	2,652,600	48	762	4,585,000
17	544	2,990,000	49	788	4,642,500
18	524	4,282,500	50	668	4,427,500
19	529	2,960,000	51	742	5,005,000
20	601	4,310,000	52	635	4,515,000
21	758	4,765,000	53	706	4,205,000
22	674	4,870,000	54	620	4,195,000
23	705	5,520,000	55	342	1,950,000
24	794	5,230,000	56	727	3,285,000
25	728	5,725,000	57	641	2,787,500
26	762	5,445,000	58	667	4,805,000
27	810	4,995,000	59	749	4,187,500
28	670	4,345,000	60	652	4,780,000
29	590	4,270,000	61	568	3,945,000
30	599	4,265,000	62	656	4,125,000
31	653	4,505,000			

Each result is the average of two tests on beams 48 inches long, 6 inches wide, and 8 inches deep, tested on a 42-inch span.

TABLE 7.—*Talbot-Jones wear tests of concrete; age, 90 days*

Section No.	Loss in weight	Depth of wear	Section No.	Loss in weight	Depth of wear
	Per cent	*Inches*		*Per cent*	*Inches*
1	3.48	0.174	32	6.42	0.321
2	7.55	.378	33	5.25	.261
3	7.57	.393	34	7.50	.375
4	5.76	.288	35	5.23	.262
5	3.91	.195	36	5.48	.274
6	5.42	.271	37	4.29	.215
7	4.06	.203	38	4.00	.200
8	5.60	.280	39	5.73	.287
9	6.51	.326	40	3.46	.173
10	6.73	.335	41	3.42	.171
11	7.73	.387	42	4.16	.208
12	6.71	.334	43	7.26	.363
13	5.82	.291	44	8.29	.415
14	6.16	.308	45	6.95	.347
15	6.05	.319	47	5.50	.276
16	4.98	.248	48	4.92	.246
17	5.05	.253	49	7.06	.353
18	5.97	.299	50	5.63	.282
19	6.39	.320	51	6.11	.305
20	6.52	.327	52	5.38	.269
21	6.31	.315	53	5.78	.289
22	6.93	.347	54	5.90	.295
23	6.00	.300	55	6.87	.344
24	4.68	.234	56	5.88	.294
25	5.72	.289	57	8.14	.407
26	5.52	.276	58	4.96	.248
27	7.08	.364	59	6.78	.286
28	5.69	.285	60	5.41	.271
29	6.28	.314	61	6.78	.339
30	7.15	.350	62	6.85	.343
31	5.28	.264			

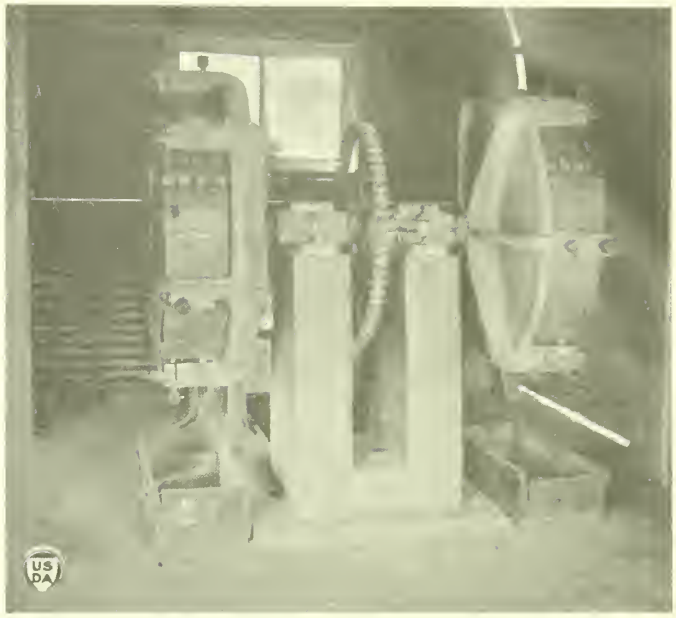
Each result is the average of three specimens.

The laboratory wear tests were made in accordance with the methods employed by Abrams [1] using the Talbot-Jones rattler, and the results are given in Table 7. A view of the rattler is shown on page 7.

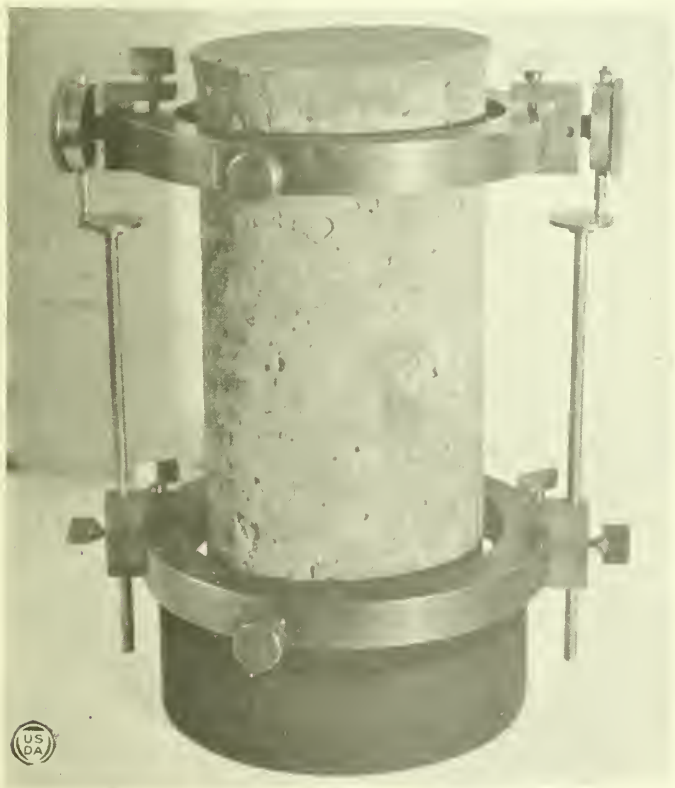
[1] Bulletin 10, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

Operation of the testing machine was begun in March, 1923, about six months after the completion of the test sections. As already noted the tests have so far been run in two stages, the first consisting of 75,000 trips of the machine equipped with solid rubber tires, and the second of 25,000 trips of the same machine, with the front wheel of each truck equipped with nonskid chains. The test with tire chains was started in July, 1923, and was continued intermittently until February, 1924, the wheels following an entirely different path from that followed during the first stage. Although a considerable portion of the second stage was run during December and January, weather conditions were so favorable that practically no snow or ice was encountered.

For the purpose of determining the actual amount of wear which took place under both rubber-tire and chain traffic, measurements were taken at four places on each test section by means of the special device shown on page 9. This device consists essentially of a rigid bar about 11 inches in length mounted on two flat, circular plates which rest upon the pavement and which are pivoted to the cross bar through universal joints. An Ames dial with a plunger capable of 1 inch extension is mounted rigidly at the center of the cross bar together with a cross bubble for leveling the apparatus. Measurements were taken at certain points on the section, the locations of which were fixed by black, circular spots of the same diameter as the bearing plates. With the instrument placed exactly on these spots, which were of course outside the line of travel, differences of reading on the dial between any two successive measurements represented the actual amount of wear which had taken place during the interval. An average of four readings on a section taken by this method is probably a fair indication of the wear on the section as long as it remains fairly uniform. Development of marked inequalities in wear, however, made this method very uncertain, so that for the final measurement under chain traffic another scheme was used. This consisted in taking a plaster cast of a portion of each section about 12 inches long, and somewhat wider than the width of the traffic groove. An average 12-inch length of each section was selected for this purpose. The casts were then sawed longitudinally through the center so that an exact profile of the surface of the pavement was obtained. Photographs were then made of these longitudinal sections. By blocking out everything except the area included between the horizontal line representing the original surface of the pavement and the final profile, an exact record of both the amount and character of wear was secured. The average depth of wear in inches over this 12-inch section was readily calculated by dividing the area by the length of the section.



The Talbot-Jones rattler



Two-ring, two-dial compressometer mounted

As already noted, measurements were taken with the Ames dial to determine the amount of wear taking place under the rubber-tired traffic. Table 8 gives the measurements of depth in inches for each of the 62 test sections at the end of 55,000 trips of the testing machine. Surface wear, under these conditions, it will be noted, is practically negligible, amounting at the point of maximum depth to only 0.018 inch. In fact a slight discoloration of the surface is about the only visible evidence of the traffic which has passed over it. This is true of every test section regardless of the quality of the aggregates composing the concrete. As a matter of fact, the various course aggregates did not come under test at all due to the fact that they were protected by a thin mortar top which formed the finished surface of each section. These observations lead to the conclusion that rubber-tired traffic, without impact, produces no material wear on the surface of a concrete pavement, at least as long as the protecting mortar top remains intact. It must be remembered, however, that traffic on our highways is not confined entirely to vehicles with rubber tires. There is still a certain percentage of steel-tired traffic, as well as an appreciable

amount of traffic using nonskid chains, especially in the Northern States during the winter months. Moreover, the solid-rubber tires used in these experiments were new and therefore produced no impact or abrasion such as might result from the use of badly worn or broken tires, where the rim comes in contact with the pavement.

THE SIGNIFICANCE OF THE CHAIN TRAFFIC TESTS

It is realized, of course, that chain traffic on concrete highways is not dense enough, except in isolated instances, to cause any such wear of the surface of a pavement as was produced on the test sections. Wear, however, does take place at the unprotected edges of joints and cracks, eventually causing raveling or disintegration unless carefully maintained by the use of bituminous materials. It was believed that the relative resistance offered by the various sections to the chain traffic would be a fairly reliable index of the ability of the several materials to resist the combined destructive influences of traffic, whether such influences produce wear at joints or cracks or cause surface disintegration.

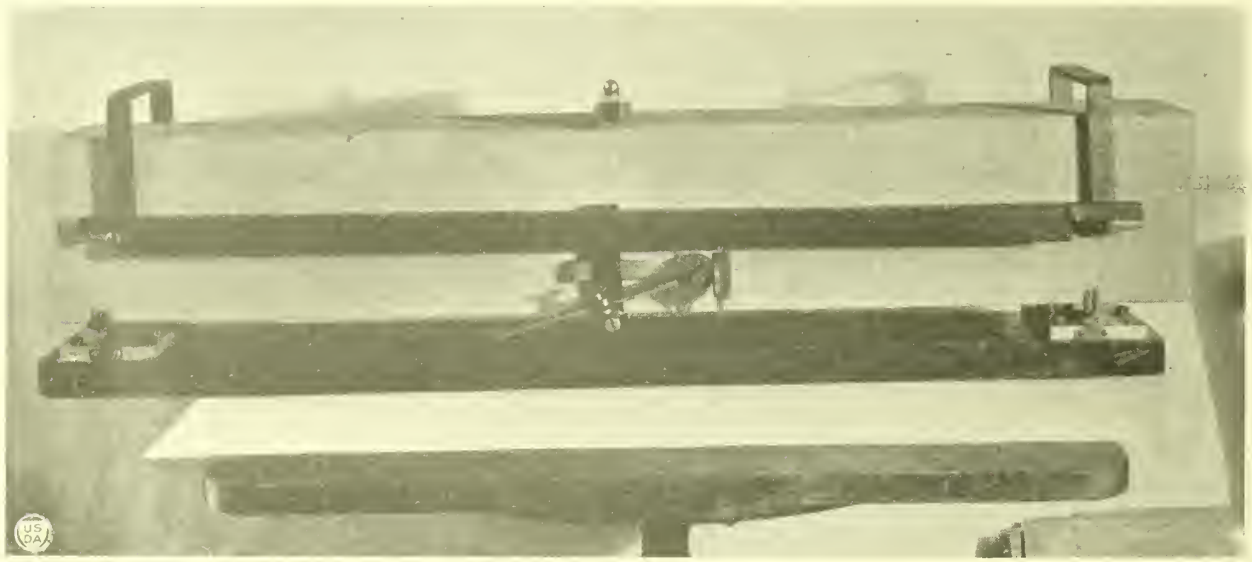
Taking the above facts into consideration, it was felt that the question was not definitely answered by simply determining that rubber tires rolling over the pavement produce no wear. It was decided therefore to continue the test, using the nonskid chains, this being not only the simplest but also the fairest method of securing the desired information.

The chains produced a noticeable effect on the surface from the beginning. Wear was rapid and progressive. Marked differences in the behavior of the different materials was also noted. In fact, at the completion of 25,000 runs, a groove almost an inch in depth had been cut in certain of the sections, whereas others showed as little as 0.2 inch wear, or only about one-fifth as much. It will be of interest to take up the several groups as outlined in Table 2 and discuss the behavior of the several sections with respect to the kind and quality of materials used in their construction.

DISCUSSION OF WEAR CAUSED BY NONSKID CHAINS

Ames-dial readings taken at various times during the test with chains are plotted in Figure 1. These results show the progress of wear from time to time. The first set of readings, for instance, was taken about the time the coarse aggregates became exposed so that it may be considered that only about 1,000 runs of the machine with chains were necessary to wear away the thin mortar top which resisted without appreciable damage over 75,000 runs of the same machine without chains.

The depth of wear at the conclusion of the second stage, together with the results of tests of the aggregates, as well as the control tests of the concrete are plotted together for purposes of comparison in Figure 2. The total depth of wear under chain traffic as well as the corresponding wear when the machine was operated without chains are also tabulated in Table 8. For the purpose of studying the relative uniformity as well as depth of wear, reproductions of the plaster-cast profiles are likewise given in Figures 3, 4, 5, and 6.



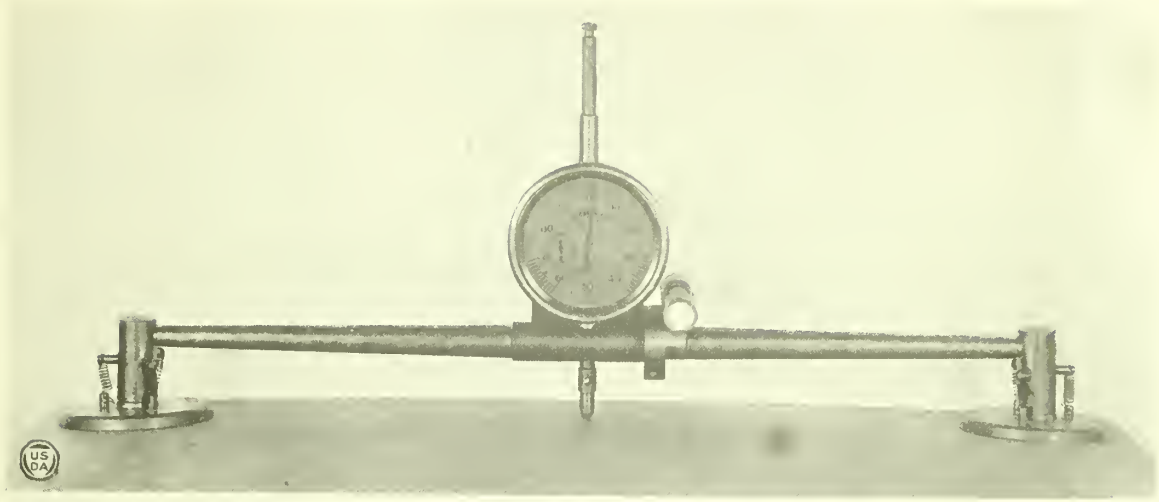
Apparatus for measuring the deflection of concrete

A discussion of the results of the wear tests, group by group, follows.

GROUP I.—SECTIONS 1 TO 10, INCLUSIVE: THE EFFECT OF QUALITY OF ROCK

Reference to Group I in Figures 2 and 3 reveals some very interesting facts. In Figure 2 the sections in Group I are arranged in the order of the resistance to wear of the various aggregates, as measured by the standard Deval abrasion test. It will be observed that the wear on the sections containing the soft limestones and sandstones was much greater than on the sections in which the harder materials were used. Sections 6, 9, and 10 in particular, the poorest materials from the standpoint of the laboratory test, show excessive wear in the pavement. On the other hand, sections 1 to 5, inclusive, all of which contain aggregate having a percentage of wear of 4 or less, show much less surface wear.

An interesting point in connection with this group is the relation between the wear on the mortar matrix



Instrument used for measuring the wear of the concrete

and the wear on the coarse aggregates. Reference to section 56, which is a 1:1½ mortar top, shows a wear of 0.33 inch, which is only a little greater than the wear on sections 2, 3, and 4 in which the hard and tough trap, quartzite, and granite were used. From the standpoint of surface wear it would seem, therefore, that it is only necessary that the coarse aggregate be at least as resistant to wear as the mortar matrix. Even a good mortar matrix, on the other hand, is apparently not able to protect a poor coarse aggregate, once it has been exposed, as witness the behavior of sections 9 and 10. Both of these sections contained extremely soft limestones which have been proposed for use in concrete roads. The results of this test, however, would indicate that stone so soft that it will show in the laboratory a percentage of wear over 7 should not be used in concrete road construction.

TABLE 8.—*Average depth of wear under rubber-tire and chain traffic*

Section No.	Depth of wear		Section No.	Depth of wear	
	Solid-rubber tires 55,000 runs	Tire equipped with chains 25,000 runs		Solid-rubber tires 55,000 runs	Tire equipped with chains 25,000 runs
	Inches	*Inches*		*Inches*	*Inches*
1	0.018	0.37	31	0.012	0.28
2	.006	.30	32	.015	.25
3	.012	.27	33	.005	.25
4	.008	.27	34	.010	.46
5	.005	.22	35	.009	.43
6	.010	.58	36	.012	.40
7	.007	.42	37	.007	.27
8	.006	.52	38	.014	.57
9	.012	.78	39	.005	.89
10	.013	.87	40	.008	.27
11	.010	.24	41	.014	.18
12	.013	.35	42	.013	.23
13	.008	.38	43	.010	.99
14	.009	.35	44	.006	.61
15	.008	.24	45	.014	.55
16	.008	.30	46	.012	.35
17	.007	.33	47	.013	.35
18	.012	.18	48	.015	.35
19	.009	.40	49	.014	.38
20	.008	.21	50	.009	.38
21	.016	.32	51	.011	.35
22	.011	.43	52	.005	.23
23	.013	.31	53	.004	.37
24	.005	.30	54	.011	.34
25	.009	.31	55	.008	----------
26	.008	.30	56	.009	.33
27	.012	.46	57	.009	.89
28	.008	.31	58	.012	.26
29	.006	.40	59	.006	.30
30	.011	.38	60	.014	.30
			61	.007	.44
			62	----------	.41

Another interesting point in connection with the first group is the behavior of the sandstone in section 6. This is the so-called Kettle River sandstone, which gave in the laboratory a percentage of wear of 13.0. Reference to Figure 3 will show that the wear on section 6, although deep, was exceptionally uniform. The depth of wear also is not so great in proportion to the percentage of wear in the Deval test, as in the case of the soft limestone sections. Whether this means that sandstones are better than limestones of equal hardness it is impossible to say, although as we shall see later, certain sections in the gravel group show the same relative behavior. With the regard to uniformity of wear, the moderately hard aggregates, i. e., those of about the same resistance to abrasion as the mortar matrix, seem to have all the best of it. Reference to Figure 3 and a visual examination of the sections themselves show that section 1 is somewhat smoother than either section 2, containing the trap, or section 3, containing the quartzite, and is much smoother than sections 9 and 10 in which the very soft limestones were used. There is not much choice between 1 and 4 or 5. Section 8, although worn rather deeply, is also smooth. This is an Ohio limestone with a percentage of wear of 6.0.

GROUP II.—SECTIONS 11 TO 19, INCLUSIVE: EFFECT OF QUALITY OF GRAVEL

In this group are 9 sections in which various types of gravel were used. The most noticeable fact about this group is that, although a very wide range in quality of gravel is represented, in no case was the depth of wear so great as in five of the sections in Group I. When tested by the Rea [2] abrasion test, the gravel samples ranged from 9.3 per cent to 29 per cent loss. Several of the samples consisted of pieces too small to make the abrasion test. Section 17 is a remarkable one. Although the South Carolina quartz gravel used in this section showed a percentage of wear of 29 (which is roughly equal to about 10 in the Deval abrasion test), the depth of wear is but little above the average for all

[2] U. S. Department of Agriculture Bulletin 949, p. 3.

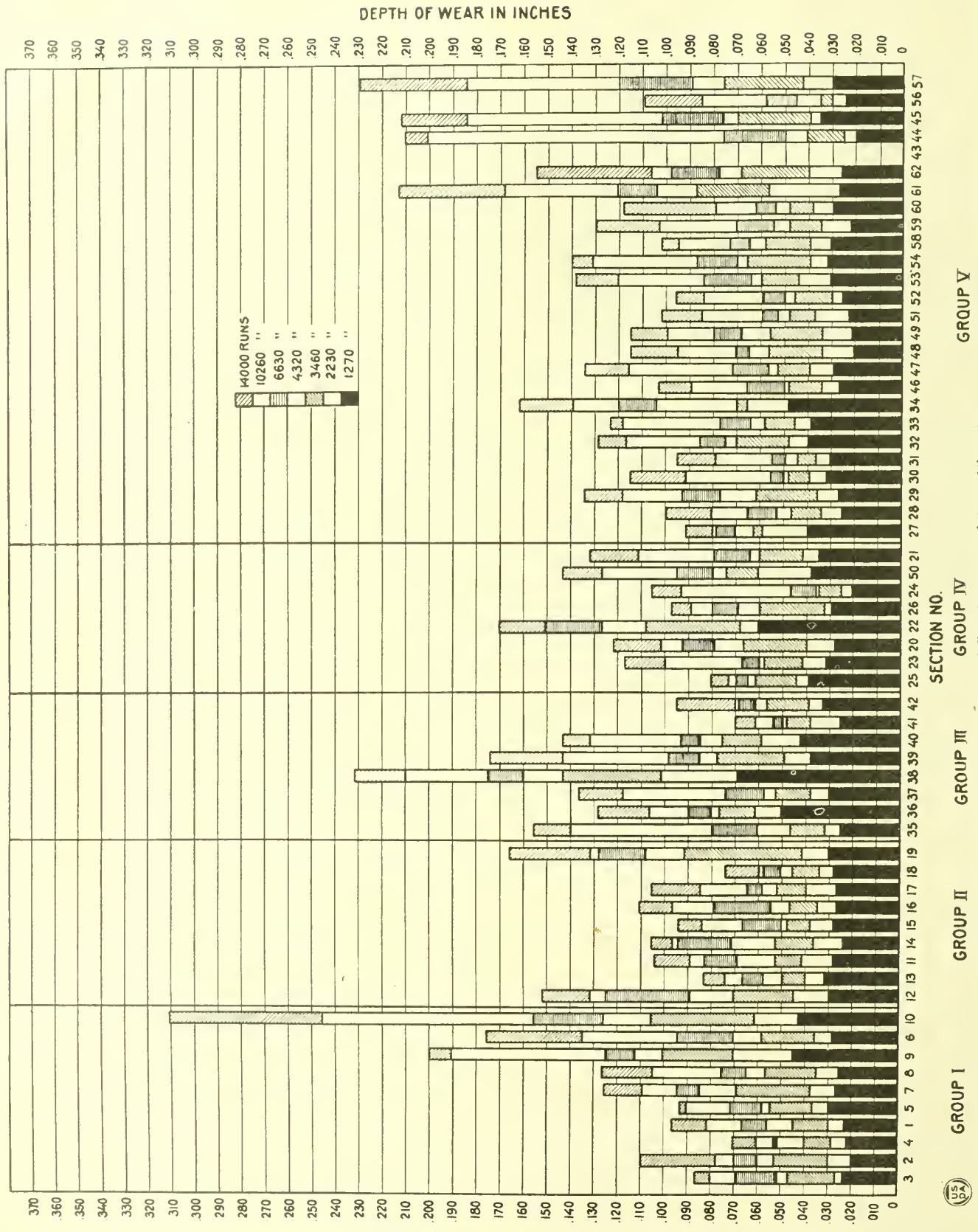


FIG. 1.—The depth of wear of all sections at several stages of the test

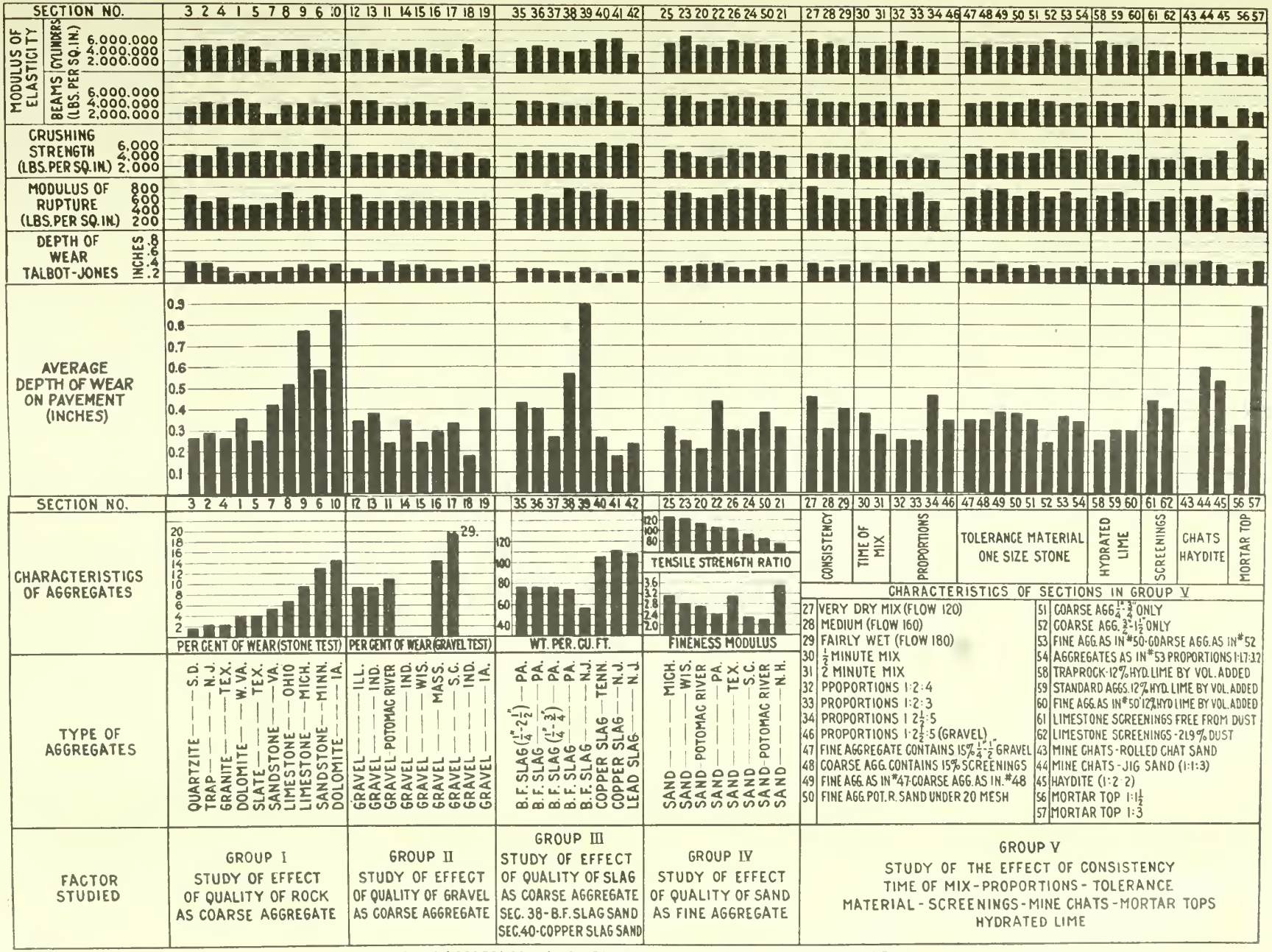

SECTION NO.
3 2 4 1 5 7 8 9 6 10
12 13 11 14 15 16 17 18 19
35 36 37 38 39 40 41 42
25 23 20 22 26 24 50 21
27 28 29
30 31
32 33 34 46
47 48 49 50 51 52 53 54
58 59 60
61 62
43 44 45
56 57
MODULUS OF ELASTICITY
CYLINDERS (LBS. PER SQ. IN.)
BEAMS (LBS. PER SQ. IN.)
6.000.000
4.000.000
2.000.000
CRUSHING STRENGTH (LBS. PER SQ. IN.)
6.000
4.000
2.000
MODULUS OF RUPTURE (LBS. PER SQ. IN.)
800
600
400
200
DEPTH OF WEAR TALBOT-JONES
INCHES
.8
.6
.4
.2
AVERAGE DEPTH OF WEAR ON PAVEMENT (INCHES)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
SECTION NO.
CHARACTERISTICS OF AGGREGATES
PER CENT OF WEAR (STONE TEST)
PER CENT OF WEAR (GRAVEL TEST)
29.
WT. PER. CU. FT.
TENSILE STRENGTH RATIO
FINENESS MODULUS
CONSISTENCY
TIME OF MIX
PROPORTIONS
TOLERANCE MATERIAL ONE SIZE STONE
HYDRATED LIME
SCREENINGS
CHATS HAYDITE
MORTAR TOP
TYPE OF AGGREGATES
QUARTZITE — S.D.
TRAP — N.J.
GRANITE — TEX.
DOLOMITE — W.VA.
SLATE — TEX.
SANDSTONE — VA.
LIMESTONE — OHIO
LIMESTONE — MICH.
SANDSTONE — MINN.
DOLOMITE — IA.
GRAVEL — ILL.
GRAVEL — IND.
GRAVEL POTOMAC RIVER
GRAVEL — IND.
GRAVEL — WIS.
GRAVEL — MASS.
GRAVEL — S.C.
GRAVEL — IND.
GRAVEL — IA.
B.F. SLAG (1" - 2 1/2") — PA.
B.F. SLAG — PA.
B.F. SLAG (1/4" - 3/4") — PA.
B.F. SLAG — PA.
B.F. SLAG — N.J.
COPPER SLAG — TENN.
COPPER SLAG — N.J.
LEAD SLAG — N.J.
SAND — MICH.
SAND — WIS.
SAND POTOMAC RIVER
SAND — PA.
SAND — TEX.
SAND — S.C.
SAND POTOMAC RIVER
SAND — N.H.
CHARACTERISTICS OF SECTIONS IN GROUP V
27 VERY DRY MIX (FLOW 120)
28 MEDIUM (FLOW 160)
29 FAIRLY WET (FLOW 180)
30 1/2 MINUTE MIX
31 2 MINUTE MIX
32 PPOPORTIONS 1:2:4
33 PROPORTIONS 1:2:3
34 PROPORTIONS 1 2 1/2 : 5
46 PROPORTIONS 1:2 1/2 :5 (GRAVEL)
47 FINE AGGREGATE CONTAINS 15% 1/4" - 1/2" GRAVEL
48 COARSE AGG. CONTAINS 15% SCREENINGS
49 FINE AGG. AS IN #47-COARSE AGG. AS IN. #48
50 FINE AGG. POT. R. SAND UNDER 20 MESH
51 COARSE AGG 1" - 3/4" ONLY
52 COARSE AGG. 3/4" - 1 1/2" ONLY
53 FINE AGG. AS IN #50-COARSE AGG. AS IN #52
54 AGGREGATES AS IN #53 PROPORTIONS 1:1.7:3.2
58 TRAPROCK-12% HYD. LIME BY VOL. ADDED
59 STANDARD AGGS. 12% HYD. LIME BY VOL. ADDED
60 FINE AGG. AS IN #50 12% HYD LIME BY VOL. ADDED
61 LIMESTONE SCREENINGS FREE FROM DUST
62 LIMESTONE SCREENINGS - 21.9% DUST
43 MINE CHATS - ROLLED CHAT SAND
44 MINE CHATS - JIG SAND (1:1:3)
45 HAYDITE (1:2 2)
56 MORTAR TOP 1:1 1/2
57 MORTAR TOP 1:3
FACTOR STUDIED
GROUP I
STUDY OF EFFECT OF QUALITY OF ROCK AS COARSE AGGREGATE
GROUP II
STUDY OF EFFECT OF QUALITY OF GRAVEL AS COARSE AGGREGATE
GROUP III
STUDY OF EFFECT OF QUALITY OF SLAG AS COARSE AGGREGATE
SEC. 38 - B.F. SLAG SAND
SEC. 40 - COPPER SLAG SAND
GROUP IV
STUDY OF EFFECT OF QUALITY OF SAND AS FINE AGGREGATE
GROUP V
STUDY OF THE EFFECT OF CONSISTENCY
TIME OF MIX - PROPORTIONS - TOLERANCE
MATERIAL - SCREENINGS - MINE CHATS - MORTAR TOPS
HYDRATED LIME
COARSE AGGREGATE, MARTINSBURG DOLOMITE UNLESS OTHERWISE NOTED
FINE AGGREGATE, POTOMAC RIVER SAND UNLESS OTHERWISE NOTED

FIG. 2.—Showing the characteristics of the sections in Groups I to V

the gravel sections and is not so great as in many of the stone sections. A close examination of this gravel shows that it is made up entirely of quartz pebbles of varying hardness, on the inside, but all protected by a hard, exterior shell. A pebble when struck even a light blow will fly to bits, yet it is very difficult to scratch the surface with a knife. Apparently the quality of hardness was more important than toughness in resisting surface wear on these test sections, which leads to the conclusion that the modified Deval test for gravel may be too severe. Because of the presence of the large shot in the rattler, it is distinctly a test for toughness. The worst gravel from the standpoint of depth of wear is the one in section 19. This gravel contained about 5 per cent of shale particles and was submitted as an example of an aggregate of doubtful quality.

While the gravel sections appear to advantage in comparison with the stone sections from the standpoint of depth of wear, comparison as to uniformity of wear is not favorable to them. Sections 15 and 16, both containing glacial gravels, show up best in this respect. It is an interesting fact in this connection that the 4 sections in which limestone is a constituent material of the gravel are the poorest as regards uniformity of wear. In sections Nos. 13 and 14, the gravels contain quite an appreciable amount of soft sandstone pebbles which have been worn away by the action of the chains, leaving marked depressions in the surface. The result is well illustrated in Figure 3. On the other hand, the mere fact that the fragments are smooth and rounded appears to make little difference as far as wear is concerned. Section 18 is a good example. Before the chain traffic was well under way, the prediction was freely made that the smooth, rounded pebbles composing this sample would speedily kick out. As a matter of fact, this is one of the best of the gravel sections, especially as regards depth of wear. Summing up, it would appear from these tests that gravel as a class should prove as satisfactory for aggregate in concrete as crushed stone, provided it is free from an excess of soft or disintegrated pebbles. Moreover, the mere fact that the particles lack toughness is no drawback provided they are sufficiently hard to resist abrasion.

GROUP III.—SECTIONS 35 TO 42, INCLUSIVE: EFFECT OF QUALITY OF SLAG

The sections in Group III were arranged with the object of studying a number of factors which are involved in the selection of slag as concrete aggregate. In the first place it was of interest to determine the relative wear of slag concrete in which the maximum size of the coarse aggregate varied as follows: 2½ inches in section 35, 1½ inches in section 36, and three-fourths inch in section 37. The quality of the slag, i. e., weight per cubic foot, remained constant at 74 pounds in these sections. In section 38, the same slag, graded 1½ inches to one-fourth inch, but combined with slag sand in place of Potomac River sand was used. Section 39, however, contained slag weighing only 56 pounds per cubic foot. Copper and lead smelter slag, with both Potomac River and copper slag sand, were employed in the construction of sections 40 to 42, inclusive. Comparing, first, sections 35 to 37, it is observed that considerably less wear has taken place on the section containing the three-fourths-inch maximum size. In fact, the total wear on this section was about the same as on the sections containing the best quality of stone and gravel, which is interesting in view of the impression that slag is not suitable for concrete road construction under any circumstances.

Section 39, however, in which the 56-pound slag was used, tells a different story, the wear being over three times as great as that on section 37. A comparison of the behavior of these sections illustrates very clearly the value of keeping the light and porous material in blast furnace slag down to a minimum. In the light of these experiments, it would seem that the requirement of 70 pounds per cubic foot used in a good many States is about correct. Blast furnace slag sand as a substitute for natural sand was tried as the fine aggregate in section 38. The results were not very encouraging, due probably to the fact that the extreme harshness of the concrete made it very difficult to finish properly. The total wear on this section was 0.57 inch or twice as great as on the corresponding section using Potomac River sand. As might be expected, the copper and lead smelter slag used in sections Nos. 40 to 42 inclusive, showed very little wear. Section 41 showed less wear than any of the 62 test sections. There would seem to be no reason why such materials as these should not make excellent aggregates for concrete roads so far as durability is concerned. Unfortunately, however, the great weight of smelter slags, involving high transportation costs, limits their use to a great extent. In conclusion, it may be said that the slag experiments indicate: (1) That somewhat better results are secured by the use of the smaller sizes; (2) that the weight per cubic foot should be not less than 70 pounds; (3) that blast furnace slag sand does not make a satisfactory substitute for natural sand; and (4) that copper and lead smelter slag form excellent aggregates so far as resistance to wear is concerned.

GROUP IV.—SECTIONS 20 TO 26, INCLUSIVE, AND SECTION 50: QUALITY OF SAND

In sections 20 to 26, inclusive, as well as section 50, variations in wear, due to both quality and grading of natural sand as fine aggregates were studied. Sands were selected for this purpose varying in strength ratio from 126 per cent to 70 per cent of standard Ottawa sand and from 2.2 to 3.5 in fineness modulus. Of this group, section 22, in which a river sand from Fayette County, Pennsylvania, was used, developed the greatest depth of wear, a little over 0.4 inch. This sand contained an appreciable amount of carbonaceous shale which probably accounts for the increased wear. It was also a rather fine sand, only 25 per cent being retained on the No. 20 sieve. The next greatest wear took place on section 50, which contained the fine Potomac River sand. Reference to Table 4 will show that only 9 per cent of this sand was coarser than a No. 20 sieve, and that the strength ratio at 28 days was only 84 per cent. Four of the test sections, Nos. 21, 14, 25, and 26, all showed about the same resistance to wear, about 0.3 inch. With the exception of No. 24, all of these may be classed as coarse sands, that is they contain more than 30 per cent of particles coarser than No. 20 mesh. The sand in No. 24, which was from South Carolina, though considerably finer, with only 17 per cent on the 20-mesh sieve, was of exceptionally good quality, being composed entirely of pure quartz grains. A comparison of this sand with that used in section 22 is of interest because they have approximately the same grading, indicating that the variation in wear is probably due to the difference in the quality of the material. Section 20, Potomac River sand, and section No. 23, contain-

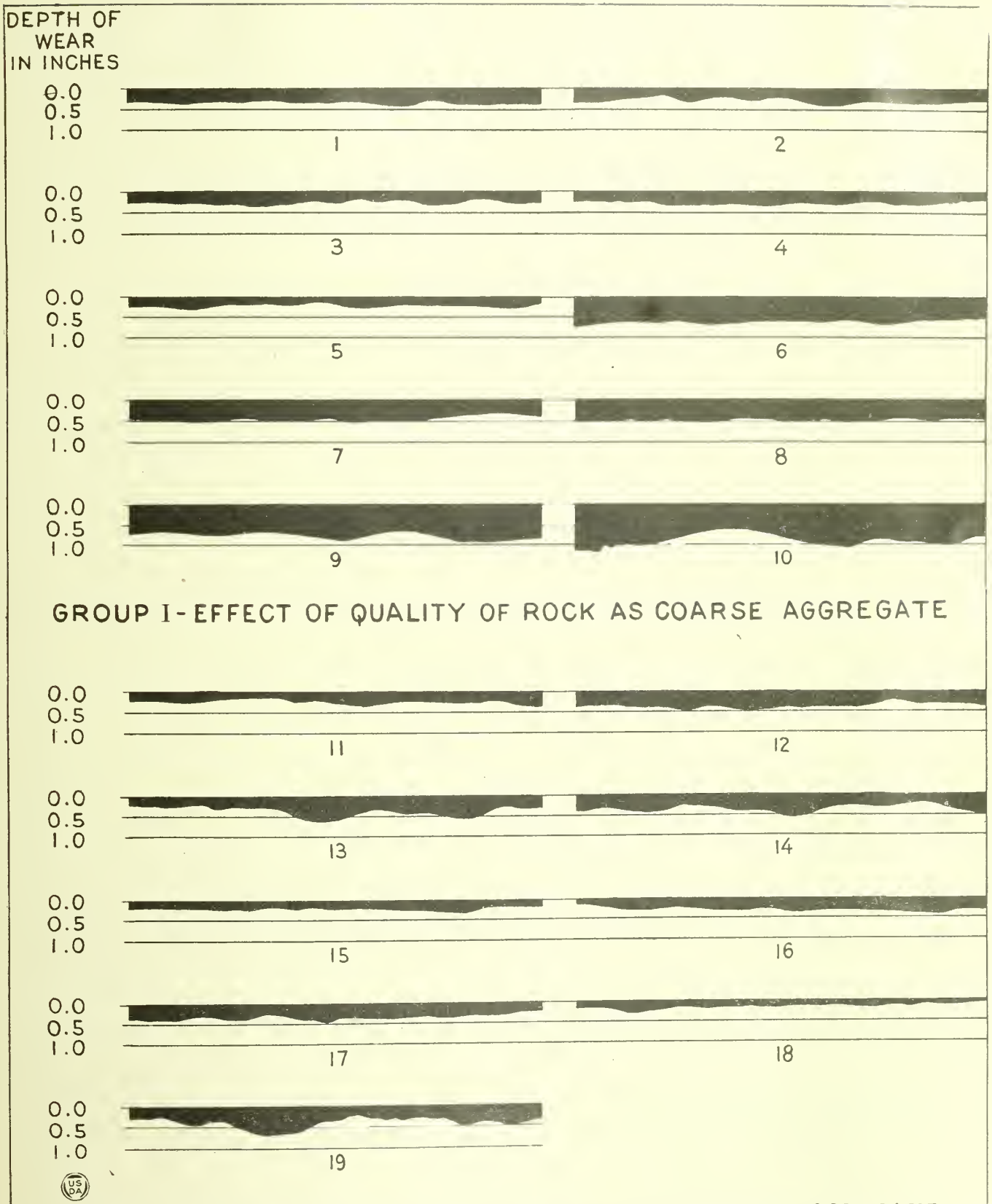


FIG. 3.—Cross sections of casts made from sections in Groups I and II

95633—24——3

ing a glacial sand from Marathon county, Wisconsin, showed the smallest amount of wear. Both of these would be classed as coarse concrete sands.

The tests in general indicate the value of the coarse particles in reducing wear. For example, compare sections 20 and 50, in which Potomac River sand was used. The materials in these sections were identical except that the sand in section 50 was finer, the proportion of particles retained on the 20-mesh sieve being reduced from 40 per cent to 9 per cent. The effect on wear is quite marked.

When we come to consider the bearing which the strength-ratio test has on the pavement wear, we have difficulty in establishing any relation. The wide range in briquette strength is not reflected in the resultant wear any more than in the concrete strength of the materials. (Reference to Figure 2 will illustrate.) This is probably due to the fact that the factors which influence the strength, i. e., gradation and quality of sand grains, do not affect the surface wear to the same degree. For instance the mortar strength test is made on a 1 to 3 mix and compared with an Ottawa sand mortar in the same proportion, whereas the cement-to-sand ratio in the test sections was 1 to 1½. The discrepancy in results, insofar as this test is concerned, is most marked in the case of section 21, in which a glacial sand from New Hampshire, containing quite an appreciable percentage of feldspar, was used. A strength test of this sand developed only 70 per cent of the strength of Ottawa sand in spite of the fact that the sand was very coarsely graded. Although this section has worn somewhat more than the Potomac River sand sections, the difference is not nearly so great as might be supposed from the results of the mortar strength test. It would seem, therefore, that the strength-ratio test is of questionable value as an indication of the quality of sand for concrete pavement work, at least insofar as resistance to wear is concerned.

GROUP Va.—SECTIONS 27 TO 34, INCLUSIVE, AND SECTION 46: EFFECT OF CONSISTENCY, TIME OF MIX, AND CEMENT CONTENT

In Group V*a*, three very important factors having a bearing on the quality of concrete were investigated. The effects of variation in consistency show up in a very interesting way. Section 28, which contained concrete of what might be termed average consistency, i. e., slump about 3 inches, showed considerably less wear than either section 27, which was a very dry concrete, slump about 1 inch, or the very wet mix with a 5-inch slump used in section 29. The concrete comprising section 30 was mixed only one-half minute to compare with section 31, on which a two-minute mix was tried. The wear is somewhat less for the longer period. Moreover, section 31 checks fairly closely with section 28, in which the time of mixing was one minute, indicating that not much is gained, from the standpoint of wear, by mixing more than one minute.

As to the effect of cement content on wear, the tests seem to show that for the usual mixes employed in concrete road construction, i. e., 1:1½:3, 1:2:3, and 1:2:4, the actual cement content has very little influence on wear. This is illustrated by comparing sections 28, 32, and 33. This conclusion is not surprising in view of the fact that cement in itself has very little resistance to abrasion but acts simply as a binder, so that for any mix in which the cement content is high enough to produce a strong bond, the actual wear will be proportional to the hardness of the aggregates. This further emphasizes the point brought out in the discussion of the coarse aggregates to the effect that the destructive action producing wear does so more by abrasion than by impact, and that hardness rather than toughness is the essential property in the aggregate. When we come to section 34, however, we find a different condition. Here we have a 1:2½:5 mix, a cement-to-sand ratio of 1:2½. The wear on this section is much greater than on any other in this group, indicating that a 1:2 mortar is about as lean as should be used. The 1:2½:5 gravel section, No. 46, although not showing as high an average depth of wear as section 34, wore very unevenly, verifying the indications from the tests in Groups I and II with regard to the relative uniformity of wear shown by crushed stone versus gravel.

GROUP Vb.—SECTIONS 47 TO 54, INCLUSIVE: EFFECT OF TOLERANCE MATERIAL AND ONE-SIZE STONE

There has been considerable controversy from time to time regarding the allowable limit for the so-called tolerance material in fine and coarse aggregates proposed for use in concrete pavements. Many specifications limit this amount of over or under size material to not more than 5 per cent. The sections in this group were built for the purpose of noting the effect on surface wear of allowing a 15 per cent tolerance as well as the effect of one-size instead of graded stone. Section 47 is a nominal 1:1½:3 mix in which the sand carried 15 per cent of gravel passing a one-half-inch and retained on a one-fourth-inch screen. In section 48 the stone contained 15 per cent coarse limestone screenings while in section 49 both aggregates contained 15 per cent of tolerance material. The total wear is somewhat greater on all of these sections than on the corresponding sections in which accurately screened aggregates were used. They are also somewhat rougher as may be noted by referring to Figure 5 on page 16. As would be expected, section 49 shows the greatest wear, amounting to 0.38 inch. Sections 51 and 52 showed surprising results. Section 51 contained one-fourth-inch to three-fourths-inch stone only, while in section 52 only the three-fourths-inch to 1½-inch sizes were used. The wear is noticeably less on section 52. Section 53 is the same as section 52, except that the fine Potomac River sand was used in place of the regular concrete sand. The wear is very much greater, illustrating again the marked effect of the coarser sand grains in preventing wear. Section 54 contains the same aggregates as section 53, in the proportion of 1:1.7:3.2 which is the proportion recommended by the Portland Cement Association for this aggregate grading. There is practically no difference in wear.

The tests in group V*b*, in general indicate that variations in gradation of aggregates within the limits tried do not affect the wear to any great extent, other things being equal. It must be realized, however, that from the standpoint of uniformity in workability and consequently in strength, it is very important to have uniformly graded material in a concrete road job. A harsh-working mix due to poorly graded aggregate almost invariably results in poor concrete in the field. If an attempt is made to place the concrete at proper consistency it is very hard to secure a satisfactory finish. If, on the other hand, sufficient water is added to make the mix workable, concrete greatly deficient in strength will result. The answer is to have the mix of such workability that it can be placed and finished without using excess water, and the authors believe that

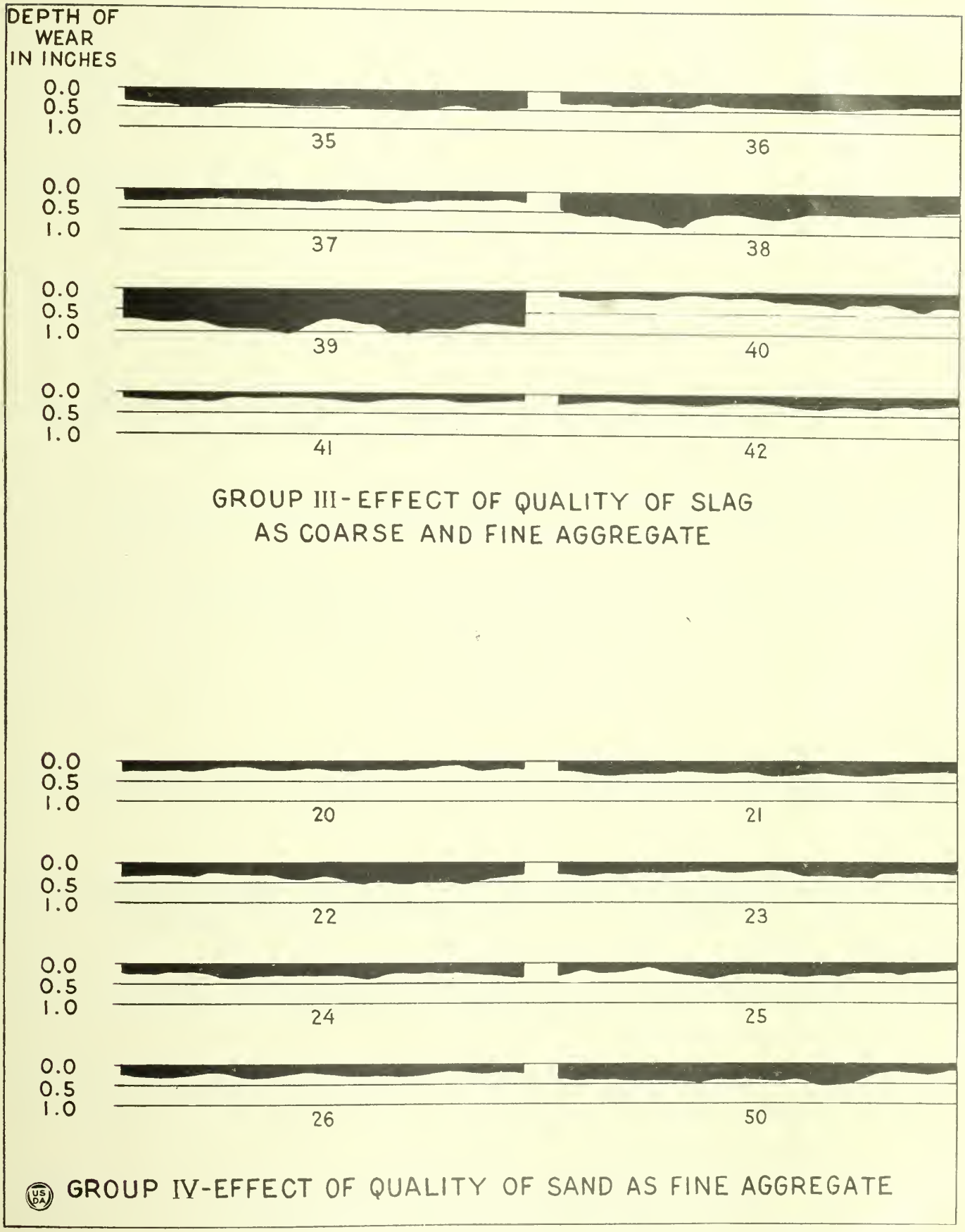


FIG. 4.—Cross sections of casts made from sections in Groups III and IV

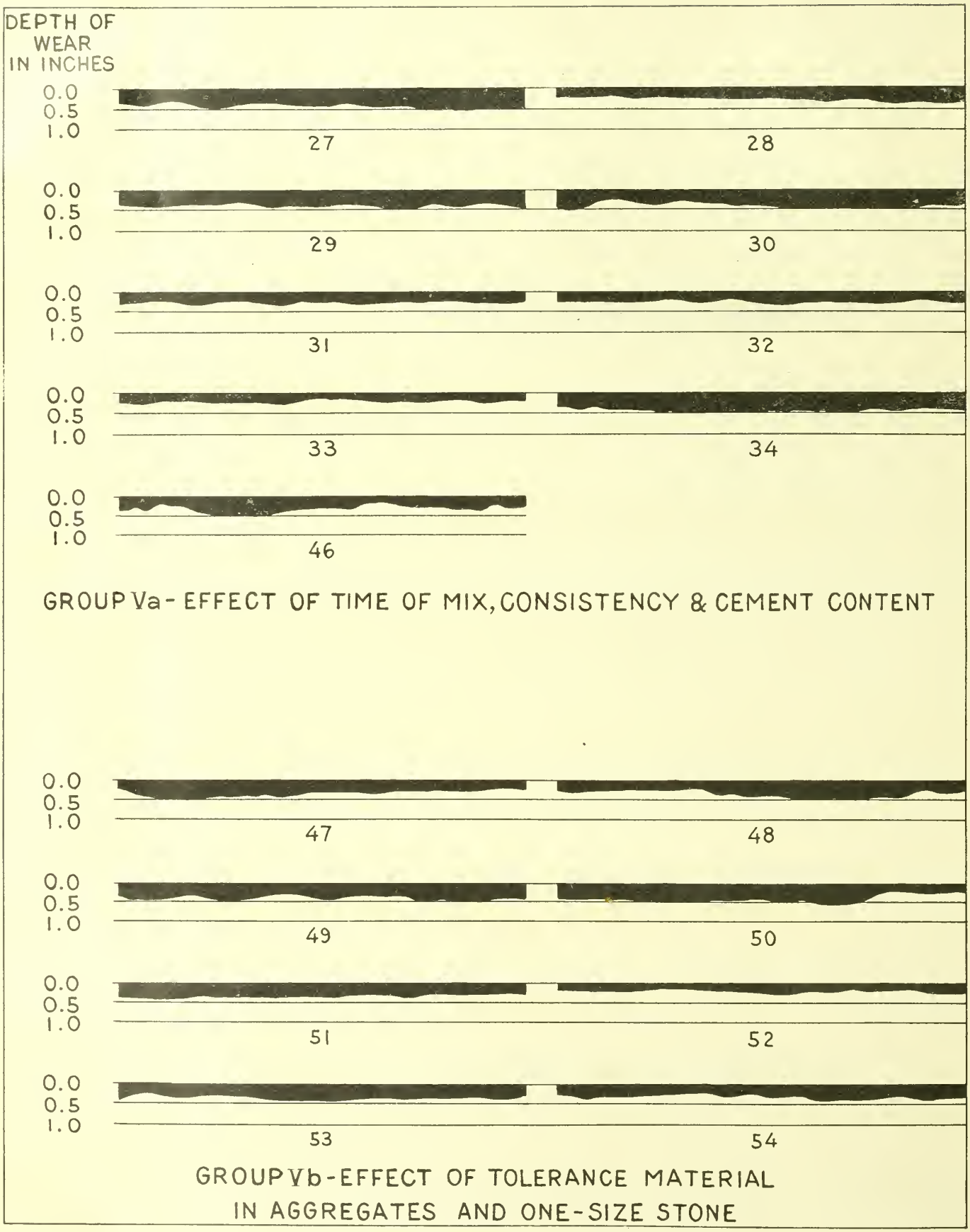


FIG. 5—Cross sections of casts made from sections in Groups V*a* and V*b*

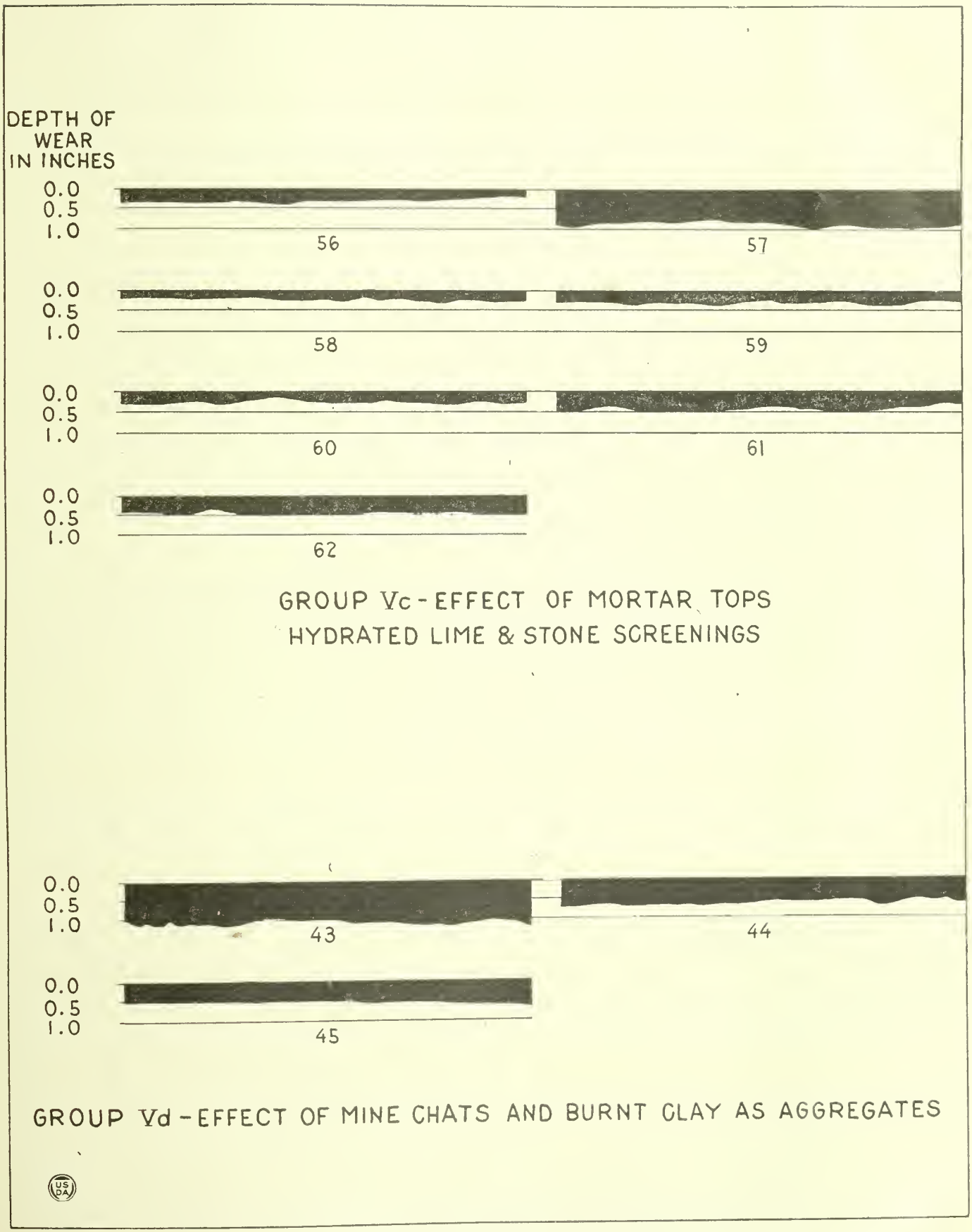


FIG. 6.—Cross sections of casts made from sections in Groups Vc and Vd

this can best be accomplished by having uniformly graded aggregates. This point is discussed here simply to emphasize a condition which is met with in practice, but which the tests did not cover, because, as has been stated, the concretes in all of these sections were of medium consistency, and not necessarily all of the same workability. These tests indicate what would probably result in practice provided the consistency were rigidly controlled irrespective of the harshness of the concrete mix.

GROUP Vc.—SECTIONS 56 TO 62, INCLUSIVE: EFFECT OF MORTAR TOP, HYDRATED LIME, AND STONE SCREENINGS

Reference has already been made to the condition of section 56 which is a 1:1½ mortar top compared with other sections in which the same mortar is combined as a matrix with several coarse aggregates. The wear of the mortar top is not much greater than that of the related concretes and is remarkably uniform. Section 57, however, which contains a 1:3 mortar has worn excessively, again emphasizing the fact that a 1:2 mortar is probably the leanest which should be used. Three sections, Nos. 58, 59, and 60, were constructed in which 12 per cent by volume of hydrated lime, based on the cement, was added to the regular 1:1½:3 mix. Section 58 contained a very hard, tough trap from Wisconsin, section 59, the Martinsburg limestone with Potomac River sand and section 60, the same stone with the fine Potomac River sand. About the only feature of interest in connection with these tests is the fact that section 60, although containing the fine sand, did not wear as much as corresponding sections in which no hydrated lime was used. Otherwise, the section behaved about the same as corresponding sections without lime. The two sections containing limestone screenings in place of sand, Nos. 61 and 62, did not behave very well. Due to the natural harshness of the concrete it was impossible to secure a proper finish, which has resulted in excessive wear. These two sections illustrate very well the danger of using screenings alone in place of sand unless the screenings are very carefully graded to give concrete which will not be excessively harsh, or unless a sufficient amount of sand is used to supply the lack of fines found usually in the commercial run of screenings.

GROUP Vd.—SECTIONS 43 TO 45, INCLUSIVE

Another illustration of the result of using harsh-working materials is furnished by sections 43 and 44 in which mine chats from Joplin, Mo., were used. Both of these sections have worn excessively, due not to the softness of the aggregates, but rather to the fact that the harsh mix prevented a proper finish from being obtained. The sections also furnish evidence indicating a lack of bond between the cement and the aggregates, due possibly to the very smooth surface of the latter which is peculiar to chert. Section 45 in which burnt clay, "Haydite," was used, likewise showed a large amount of wear, about double the average wear of those sections using standard aggregates.

THE RESULTS OF CONTROL TESTS

The control tests include determinations of the crushing strength, transverse strength, modulus of elasticity and resistance to wear as measured by the Talbot-Jones wear test. In this paper, however, the authors have been principally concerned with a discussion of the relations which exist between the wear on the test pavement sections and the physical properties of the aggregates used in them. No attempt will be made, therefore, to discuss in detail at this time the significance of the concrete tests. There are, however, a few salient points to which it may be well to call attention. By referring to Figure 2, it is immediately apparent that there is no consistent relation between the results of either the compression or the transverse strength tests and the depth of pavement wear. This is particularly noticeable in connection with the study of rock as coarse aggregate in Group 1. Although the crushing strength of the concrete in this group varies considerably, there is no relation whatever between this property and the depth of wear. The average crushing strength of the concrete in sections 8, 9, and 10 is actually higher than in concrete in which the very hard aggregates were used.

Nor do we find any relation between wear and strength in the gravel group. While the crushing strength of the concrete in section 19 is low, its modulus of rupture is practically identical with the other concretes in this group. In the case of slag the same condition holds true. Section 39, containing the light slag, in fact, has a higher modulus of rupture than the corresponding sections containing the heavy slag. An interesting point in connection with slag is the high crushing strength developed by the smelter slag. These high values, however, are not reflected in the transverse tests. About the only feature of interest in connection with the sand Group IV, is the fact that the sand showing the greatest wear has likewise the lowest crushing strength. The results of the control tests so far as they reflect variations in consistency, time of mix and cement content, shown in Group V*a*, are fairly consistent. Variations in gradation, shown in Group V*b*, however, apparently have little effect on the strength, at least as far as is indicated by the limited number of tests made. The variation in both crushing and transverse strength is high but not consistent.

The results of the Talbot-Jones wear test likewise appear to give little indication of the actual resistance of the concrete to wear produced by traffic. The authors believe that this is due to the fact that the Talbot-Jones test is not strictly an abrasion test. The shot in the rattler have an appreciable shattering effect, especially at the edge of the block, which is not duplicated in actual traffic.

CONCLUSIONS

Before attempting to draw general conclusions from these tests, it may be well to restate certain premises regarding the volume and character of traffic to which the test pavement has been subjected. These premises follow:

1. That on the average 18-foot concrete road, approximately 10 per cent of the total traffic moving in both directions passes over a band 6 inches in width at the point of maximum concentration. Therefore, the traffic passing over the test road should be multiplied by at least 10 to obtain the equivalent volume of traffic on an actual pavement.

2. That the comparative resistance of each test section to the surface wear produced by the tire chains is an indication of the comparative resistance which would be offered by the concrete to wear or disintegration at the edges of exposed joints and cracks under service conditions.

3. That conditions which cause surface wear, such as steel tires, tire chains tracking through snow, etc., are present to such an extent as to make it necessary to give the question of wear consideration in the selection of concrete aggregates.

The authors believe that the data so far secured warrant the following conclusions:

1. That the rate of wear of stone concrete is, in general, not affected by the coarse aggregate, provided the coarse aggregate is equal or superior to the mortar matrix in resistance to wear.

2. That excessive wear will result from the use of very soft stone as coarse aggregate even though used in conjunction with a mortar of satisfactory quality. From the results of these comparative tests, it would appear that stone with a percentage of wear over 7 should not be used in concrete road construction.

3. That gravel concrete, in general, is at least as satisfactory from the standpoint of wear as stone concrete.

4. That gravels consisting essentially of siliceous materials are superior as regards both the amount and uniformity of wear to those containing a preponderance of calcareous fragments.

5. That gravels consisting of rounded particles are as satisfactory from the standpoint of wear as those consisting either wholly or in part of angular or crushed fragments.

6. That small amounts of shale occurring in the coarse aggregate will cause both excessive and uneven wear.

7. That the modified abrasion test for gravel in its present form is not an indication of the wear-resisting properties of coarse aggregates. It is suggested that if the severe impact action of the steel balls were decreased, much more indicative results would be secured.

8. That blast furnace slags should prove satisfactory for use in concrete pavements provided the proportion of light, porous slag is so controlled that the weight per cubic foot will be at least 70 pounds.

9. That the presence of large amounts of light, porous fragments in blast furnace slag will cause excessive wear.

10. That somewhat better results are secured by the use of the smaller sizes of slag.

11. That slag or stone screenings are, in general, unsatisfactory as substitutes for natural sand as fine aggregates in concrete road construction.

12. That the copper and lead smelter slags used in these tests would make satisfactory aggregates for concrete road construction from the standpoint of wear.

13. That coarse sands, other things being equal, show greater resistance to wear than fine sands.

14. That the so-called "tensile-strength-ratio" test is no indication of the wear-resisting properties of concrete made with these sands.

15. That the Talbot-Jones wear test is not, in general, an indication of the wear which takes place under traffic.

16. That neither the crushing nor the transverse strength of concrete is a measure of its wear-resisting properties.

17. That the addition of hydrated lime in the proportion used in these tests does not affect the wear-resisting properties of concrete.

18. That so far as resistance to wear alone is concerned, increasing the cement content beyond a cement-sand ratio of 1:2 does not materially affect the concrete. Leaner mixes on the other hand show marked increase in wear.

19. That unusual precautions should be taken in using mine chats or other similar harsh-working materials, so as to increase workability to a maximum and thus make possible a smoother surface finish.

20. That, other things being equal, either an excessively dry or an excessively wet mix will show less resistance to wear than concrete of medium consistency.

ROAD BOND ISSUES IN RELATION TO TOTAL DEBT

A COMPARISON OF THE HIGHWAY INDEBTEDNESS OF STATES AND LOCAL SUBDIVISIONS WITH THEIR TOTAL DEBT

By HENRY R. TRUMBOWER, Economist, U. S. Bureau of Public Roads

THE attention of the public has frequently been called to the amount of outstanding bonds which the States and their several subdivisions have issued from time to time to meet public expenditures. It has been intimated that a very substantial part of this public indebtedness consists of bonds issued to meet the costs of highway construction and improvements. An inquiry made by the United States Bureau of Public Roads into the financial status of our rural highways and the recent reports of the United States Census Bureau relative to State and local taxes and public indebtedness furnish the basis for a determination of the faets. An examination and analysis of these data prepared by governmental agencies show that for the country as a whole the indebtedness incurred on account of rural highway expenditures accounts for but a small part of the public indebtedness to which the States and their subdivisions have obligated themselves.

At the end of 1921 the total amount of highway bonds outstanding was $1,222,312,300. Of the whole issue 72 per cent are represented by local bonds, the obligations of counties, townships, and districts; 28 per cent are the obligations of State governments, as shown by the following table:

	Amount	Per cent
State Bonds	$345,574,100	28
Local Bonds	876,738,200	72
Total	1,222,312,300	100

There are only two States in the whole country which have no highway bonds of any kind outstanding, State or local. These are North Dakota and Vermont. All the other States have local bonds outstanding, with the

exception of Colorado which has issued State highway bonds but has no local bonds issued for that purpose.

Over half of the States, 27 in number, have no State highway bonds outstanding. These States are:

Alabama	Kentucky	Ohio
Arizona	Louisiana	Oklahoma
Arkansas	Minnesota	South Carolina
Connecticut	Mississippi	Tennessee
Florida	Missouri	Texas
Georgia	Montana	Vermont
Indiana	Nebraska	Virginia
Iowa	New Jersey	Washington
Kansas	North Dakota	Wisconsin

New York has the largest issue outstanding, both State and local, amounting to $121,681,100; California is a close second with $101,258,000.

COMPARISON OF HIGHWAY BONDS AND TOTAL INDEBTEDNESS

The United States Census Bureau has reported the total amount of bonds issued for all purposes by States, counties, and all other subdivisions (including cities, villages, townships, and school districts). For the year 1922, which is the closest information which can be compared with the highway indebtedness, the total indebtedness, less the sinking fund assets set aside to meet such debts, was $8,695,906,000. This total indebtedness, which may be regarded as the net amount, is divided as follows:

	Amount	Per cent
State	$936,414,000	10.5
Counties	1,255,226,000	14.5
All other subdivisions	6,504,266,000	75.0
Total	8,695,906,000	100.0

It will be observed that by far the major part of this total debt, or 75 per cent, is the obligation of cities, villages, townships, and school and other districts; the State and county share was but 25 per cent.

In order to make a closer comparison with the indebtedness for highway purposes, the debt of the counties and all other subdivisions, amounting to $7,759,492,000, may be called local. The direct comparison follows:

	Total indebtedness	Highway bonds	
		Amount	Per cent of total debt
State	$936,414,000	$345,574,100	37
Local	7,759,492,000	876,738,200	11
Total	8,695,906,000	1,222,312,300	14

According to this it is shown that the total highway bonds of the country amounted to 14 per cent of the total indebtedness of the States, counties, and other political subdivisions.

At the end of the fiscal year, June 30, 1923, the debt of the Federal Government was $22,525,773,000. Adding to this amount the State and local indebtedness we get $31,221,679,000 as the total indebtedness of the country. Of this amount $1,222,312,300 was represented by highway bonds, or 3.9 per cent.

A comparison of the amount of highway bonds outstanding and the total debts of States and their local subdivisions is presented below, according to geographic divisions and States:

	Total debt	Highway bonds	
		Amount	Per cent of total debt
NEW ENGLAND STATES			
Maine	$42,457,000	$6,439,300	15.2
New Hampshire	16,604,000	570,100	3.4
Vermont	11,994,000		
Massachusetts	326,245,000	26,820,800	8.2
Rhode Island	49,893,000	2,952,000	5.9
Connecticut	100,954,000	[1] 400,000	.4
Total	548,147,000	37,182,200	6.8
MIDDLE ATLANTIC STATES			
New York	$1,683,820,000	$121,681,100	7.2
New Jersey	382,001,000	25,833,000	6.7
Pennsylvania	550,365,000	[2] 50,000,000	9.1
Total	2,616,186,000	197,514,100	7.5
EAST NORTH CENTRAL STATES			
Ohio	$670,338,000	$70,936,500	10.4
Indiana	152,840,000	62,415,900	41.0
Illinois	364,019,000	20,617,300	5.6
Michigan	361,779,000	55,234,200	15.3
Wisconsin	104,523,000	10,201,600	9.8
Total	1,653,499,000	219,405,500	13.3
WEST NORTH CENTRAL STATES			
Minnesota	$269,607,000	$21,402,600	8.0
Iowa	151,911,000	25,677,800	16.8
Missouri	118,276,000	14,942,600	12.6
North Dakota	40,266,000		
South Dakota	50,554,000	3,649,000	7.2
Nebraska	97,819,000	2,848,000	2.9
Kansas	123,470,000	9,182,400	7.4
Total	851,903,000	77,702,400	9.1
SOUTH ATLANTIC STATES			
Delaware	$22,453,000	$7,144,000	31.8
Maryland	120,954,000	18,212,900	15.2
Virginia	119,494,000	14,936,300	12.5
West Virginia	70,512,000	37,727,900	53.5
North Carolina	182,711,000	55,808,500	30.6
South Carolina	65,010,000	13,081,000	20.0
Georgia	64,045,000	15,659,000	24.4
Florida	98,243,000	24,130,200	24.6
Total	743,422,000	186,699,800	25.1
EAST SOUTH CENTRAL STATES			
Kentucky	$50,324,000	$7,135,800	14.2
Tennessee	142,159,000	28,693,500	20.2
Alabama	75,189,000	8,758,500	11.6
Mississippi	111,499,000	37,912,400	34.0
Total	379,171,000	82,500,200	21.8
WEST SOUTH CENTRAL STATES			
Arkansas	$91,279,000	$50,955,700	55.7
Louisiana	126,946,000	28,571,400	22.5
Oklahoma	129,977,000	11,547,600	8.9
Texas	356,342,000	96,517,900	27.1
Total	704,544,000	187,592,600	26.6
MOUNTAIN STATES			
Montana	$65,229,000	$9,775,800	14.9
Idaho	61,693,000	19,772,200	32.0
Wyoming	19,128,000	3,745,000	19.6
Colorado	99,645,000	2,000,000	2.1
New Mexico	25,010,000	2,001,100	8.0
Arizona	44,973,000	18,501,000	41.2
Utah	50,041,000	11,785,200	23.6
Nevada	7,861,000	1,438,000	18.3
Total	373,580,000	69,018,300	18.5
PACIFIC STATES			
Washington	$169,063,000	$20,547,900	12.1
Oregon	137,177,000	42,891,300	31.2
California	519,214,000	101,258,000	19.5
Total	825,454,000	164,697,200	19.8
RECAPITULATION			
New England States	$548,147,000	$37,182,200	6.8
Middle Atlantic States	2,616,186,000	197,514,100	7.5
East North Central States	1,653,499,000	219,405,500	13.3
West North Central States	851,903,000	77,702,400	9.1
South Atlantic States	743,422,000	186,699,800	25.1
East South Central States	379,171,000	82,500,200	21.8
West South Central States	704,544,000	187,592,600	26.6
Mountain States	373,580,000	69,018,300	18.5
Pacific States	825,454,000	164,697,200	19.8
Total	8,695,906,000	1,222,312,300	14.0

[1] Complete data not available. [2] No data for local bonds.

WHITE JOINT FILLER FOR CONCRETE ROADS

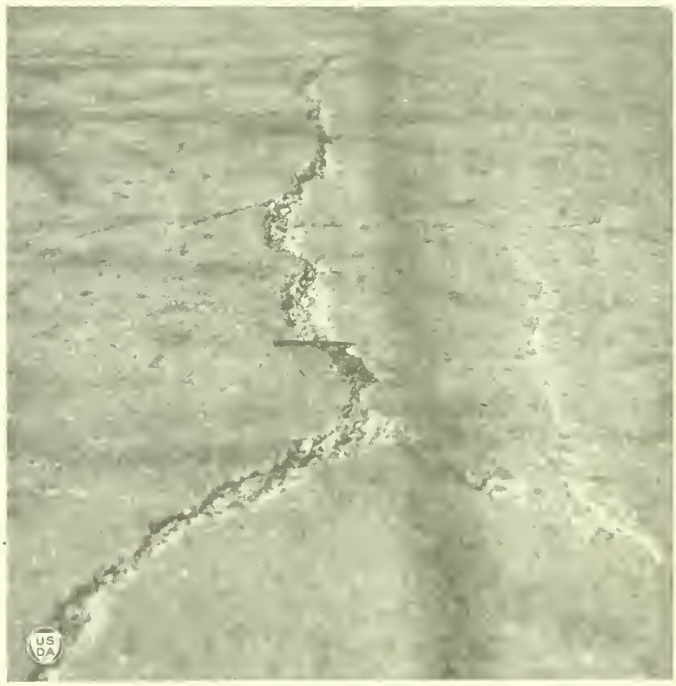
TWO-YEARS' TRIAL SHOWS NEW MATERIAL HAS DECIDED VALUE

By L. G. CARMICK, Chemist, U. S. Bureau of Public Roads

TWO years' trial of a light-colored material developed by the Bureau of Public Roads for use as a filler for joints and cracks in concrete roads has demonstrated that the material has qualities which make it decidedly valuable for the purpose. Considering the lasting qualities of the material it has also been demonstrated that it is certainly not much more costly than other fillers, and it is possible that it will be found to be no more expensive.

In the early part of 1922 the Bureau of Public Roads began a search for some material of about the same color as Portland cement concrete that could be used for filling cracks and expansion joints in concrete roads. The bituminous materials which have always been used for this purpose are considered fairly satisfactory in most respects, but they unquestionably disfigure the road. A large number of different substances were experimented with and abandoned, which it is not worth while to mention here. However, after numerous failures a plastic material was found which seems to have most of the desired qualities.

The light-colored material fills the joints well, does not bleed, and has outlasted two fillings with asphalt



The cracks filled with asphalt have been refilled once and are now badly in need of further attention

CHARACTER OF THE MATERIAL

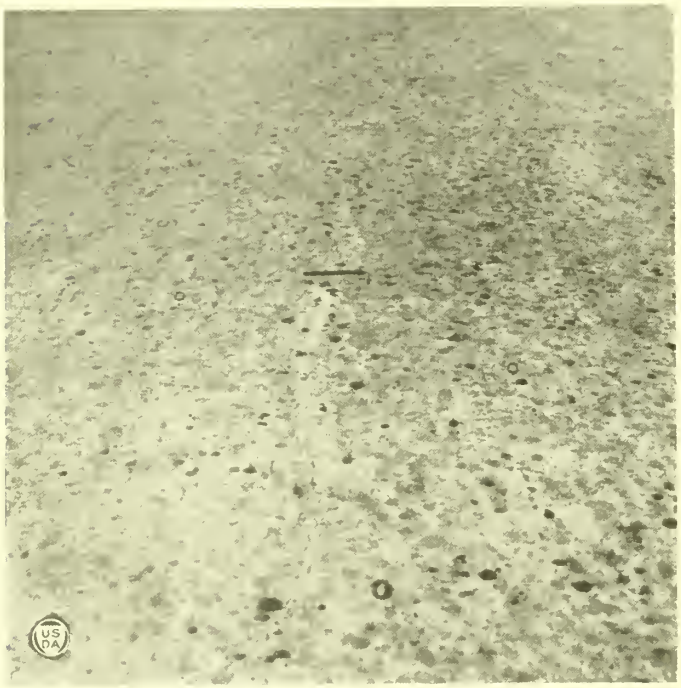
It is an intimate mixture of crude unvulcanized rubber with rosin in the proportion of one part of rubber to about ten of rosin, to which mixture is added a considerable quantity of a neutral white pigment, preferably barytes or titanox. Some difficulty was experienced at first in securing a homogeneous mixture of these materials, as rubber does not dissolve readily in melted rosin. But, eventually it was found that if the rubber were softened by allowing it to absorb about twice its weight of a light petroleum lubricating oil, and the mixture effected in a steam-jacketed iron vessel provided with revolving arms which exerted a mixing and kneading action, a smooth product could be obtained. This material differs in many respects from tar and asphalt. In color it is almost white, but it can be readily tinted to any desired shade. It is extremely sticky, especially when hot, and has a high degree of ductility. In these respects it resembles bituminous materials, but it is far less affected by temperature changes. It is more difficult to melt and even in the coldest winter weather met with in the vicinity of Washington it does not become brittle.

To use this material successfully these facts must be borne in mind. It may be melted in open kettles at the roadside, but the melting takes longer than with bitumens and care should be taken to prevent the material from catching fire and to avoid overheating. At a temperature of about 110° to 125° C. it is fluid enough to pour, and it does not become much more fluid even though the temperature be considerably raised, but the quality of the material is injuriously affected if heated much above this temperature. It was also found that it did not flow to the bottom of the cracks as readily as bituminous fillers, but this was overcome by the use of a hot iron implement, with which it can be worked down into even very narrow cracks. The tool used at present somewhat resembles a laundry iron, but with a handle long enough to enable a man to use it without stooping uncomfortably. Two

of these are necessary, one being heated while the other is in use. As soon as the material is poured into the crack it is gone over with the hot iron and in this way very good results are obtained.

BEHAVIOR AND COST OF FILLER

This crack filler has been used experimentally in several of the States and the reports have been generally favorable. It has also been used in the Columbia Pike, an experimental road constructed by the Bureau of Public Roads near Washington. Frequent inspections of the material as used in this road, and



The new material is inconspicuous and, in some cases, almost invisible

comparison with the several types of bituminous fillers also used provide the basis for the statements made in this article. On the Columbia Pike the cracks were filled with the new material in December, 1922. A recent inspection shows them to be in good condition still. The cracks are well filled and the material has shown no tendency to flow out or "bleed." Traffic has done it no harm. The cracks that were filled after the men had learned how to handle the material are particularly good. They are inconspicuous and, in some cases, almost invisible. Cracks that were filled at the same time with asphalt had to be filled again in September 1923 and are now badly in need of further attention. The asphalt flows out causing an unsightly black streak in many cases several inches wide on each side of the crack.

This light-colored crack filler costs at present about 10 cents per pound which is, of course, much more than asphalt or tar, and furthermore it is not likely that its cost can be very greatly reduced. Also the labor cost of applying it is somewhat more than for asphalt and it was accepted as a fact in the beginning that it could not compete in price with bituminous fillers. Its use was felt justified, however, because in any case it would be a very small item in the cost of a first-class concrete highway, and the finer appearance of the road, due to the absence of black streaks was considered to be worth the additional cost.

The asphalt filler has flowed out of the joints, forming unsightly black streaks several inches wide

However, in view of recent inspections it seems that its cost, taken over a period of years may not be so much greater than asphalt after all. For instance, cost data on the road mentioned show that filling cracks with asphalt cost $0.0075 per square yard of road for labor and material. The cost of the light-colored filler was about 1 cent per square yard for material and 1¼ cents for labor, the total being about three times as much as for asphalt. It is, therefore, apparent that if the light-colored material lasts three times as long as asphalt it will entail no greater expense. Experience so far seems to indicate that this will be the case.

PROTECTION OF CONCRETE AGAINST ALKALI

PROGRESS REPORT OF TWO SERIES OF TESTS BY U. S. BUREAU OF PUBLIC ROADS

WATER-GAS AND COAL TARS AND PARAFFIN TREATMENTS INVESTIGATED

By Dr. E. C. E. LORD, Petrographer, U. S. Bureau of Public Roads

TWO series of investigations by the United States Bureau of Public Roads directed toward the development of treatments for concrete to protect it against attack by alkali have yielded certain results which are worthy of notice, although the investigations are still in progress, and it is not yet possible to report definitely with regard to the relative success of the several treatments studied.

Only one form of treatment was tried in the earlier experiments, that being immersion in thin, crude water-gas tar.

The specimens employed in the first experiments consisted chiefly of sections of well seasoned, 6-inch concrete drain tile containing very little coarse aggregate, and ordinary tension and compression strength specimens made up of one part cement and three parts sand. In addition to these samples, concrete and mortar specimens were prepared for tar penetration determinations.

The tile sections, after thorough drying in air, were immersed in thin, liquid water-gas tar at room temperature for a few minutes until all air had been expelled, when they were removed from the bath and allowed to drain and dry out in air, or they were heated in the air bath at 110° C. for four hours. In some cases two treatments of this kind were given in order to assure a complete penetration of the tar. It was found that the maximum amount of tar absorbed in this way was from 7 to 8 per cent after draining for 24 hours, and about 4½ per cent when dried in the air bath.

The mortar specimens for tension and compression tests were treated in the same way, but allowed to remain in the tar for six days before drying in the air bath.

PROPERTIES OF THE WATER-GAS TAR

Tests of the tar used showed the following properties:

1.	General characteristics	Very thin liquid
2.	Specific viscosity (Engler)	3. 17 at 25° C. 1. 98 at 40° C.
3.	Total bitumen soluble in CS_2	Per cent 96. 99
4.	Free carbon	do 0. 30
5.	Inorganic matter	do 0. 05
6.	Water	do 2. 66
7.	Distillation (170° C.–300° C.)	do 42. 2
8.	Residuum (semi solid)	do 57. 8
9.	Specific gravity (25° C.)	1. 053
10.	Specific gravity of distillate (25° C.)	0. 956

In order to estimate the penetrating value of this tar, 4-inch by 2-inch sand-mortar cylinders were prepared which, after curing for seven days in damp air and drying in the laboratory for six days, were placed in glass stoppered jars containing 25 cubic centimeters of tar. After one month it was found that the tar had penetrated uniformly upward to a height of from 2¾ to 3¾ inches above the original level of the tar.

EXPOSURE TESTS

Prior investigation by the Department of Drainage and Waters of Minnesota[4] in connection with a study of the effect of alkali on concrete drain tile, had indicated in some cases a loss from the cement of about 30 per cent lime and the formation of gypsum (hydrous sulphate of lime) in large quantities. In view of these results it was decided to expose the test specimens to a solution containing 1½ per cent sodium sulphate and 1½ per cent magnesium sulphate, which approximates an average maximum concentration of these salts in alkali soil waters, and determine the amount of lime passing into solution as a measure of alkali attack.

The tests were carried out as follows: Sections of tile, both treated and untreated and weighing approximately 2,200 grams, were placed in porcelain-lined jars containing 10,000 cubic centimeters of the alkali solution and kept well covered. Determinations of lime were made from time to time and the solutions renewed at frequent intervals. Table 1 shows the loss of lime after various periods of exposure and the total lime dissolved at completion of the tests.

TABLE 1.—*Solubility of tar-treated and untreated concrete drain tile in 3 per cent alkali solution*

Sample No.	Weight of sample	Method of treatment	Time exposed	Loss of CaO	Remarks
	Grams		*Weeks*	*Grams*	
I	2,297	Immersed twice in tar and dried at 110° C.	6	0. 56	
			10	1. 31	
			14	1. 65	
			18	1. 52	
			22	2. 45	
			26	2. 10	
			30	2. 18	
			38	2. 10	
		Total		2. 45	
II	2,198	Immersed once in tar and dried at 110° C.	7	0. 61	
			12	1. 87	
			16	0. 82	Solution renewed.
			24	1. 52	
			28	2. 35	
			32	2. 04	
			36	0. 67	Solution renewed.
		Total		4. 89	
III	2,360	Immersed twice in tar and dried at 25° C.	10	1. 30	
			14	0. 56	Solution renewed.
			18	1. 58	
			22	1. 40	
			26	1. 44	
			30	3. 06	
			34	2. 61	
			38	1. 42	
		Total		4. 36	
IV	2,171	Untreated	5	4. 45	
			10	4. 38	Solution renewed.
			17	4. 02	Do.
			22	3. 96	Do.
			26	1. 79	Do.
			30	2. 29	
			38	5. 40	
			42	4. 04	
			46	1. 08	Solution renewed.
		Total		23. 29	
V	2,281	Untreated	2	2. 40	
			3	2. 19	Solution renewed.
			4	1. 25	Do.
			6	0. 50	2 weeks in tap water.
			10	4. 18	Solution renewed.
			14	2. 76	Do.
			18	1. 13	Do.
			22	1. 44	
			26	2. 40	
			30	2. 25	
			34	0. 67	Solution renewed
			38	1. 81	
		Total		17. 49	

[4] Report of Concrete-Alkali Investigations in Minnesota. State of Minnesota, Department of Drainage and Waters, E. V. Willard, Commissioner, 1919–20, pp. 13–16.

It will be seen that the lime dissolved in the untreated samples (Nos. IV and V) does not increase progressively with the time exposed, but after reaching a maximum (about 4½ grams in 5 weeks) appears to decline, owing possibly to the formation of insoluble calcium sulphoaluminate compounds. Comparing the total loss of lime in the untreated samples (IV and V) with that of the treated (Nos. I, II, and III) for the same length of time (38 weeks) it will be observed that the former have lost from 7 to 9 times as much lime as No. I, which received a double tar treatment, and from 3 to 5 times as much as the samples given but one treatment (No. II), or two treatments and dried at 25° C. (No. III). In other words, it may be said that the foregoing results indicate a 7 to 9 fold protection in the case of thoroughly impregnated material (I) and about 4 fold when insufficient tar was absorbed (II), or where the fixation of the tar residuum was incomplete before exposure to the alkali solutions (III).

TENSION AND COMPRESSION TESTS

Tension and compression tests were made on mortar treated with water-gas tar and on untreated material after remaining in 3 and 10 per cent alkali solutions and in tap water for 28 and 182 days. The results are given in Table 2.

It will be observed that while the untreated samples failed completely after 6 months in 3 per cent alkali solution (2), the treated specimens (3) gave higher values for tension and about the same for compression as those stored in tap water for the same length of time (1). Treated samples stored in 10 per cent alkali solution (6) showed a loss in tension of about 50 pounds as compared with untreated samples after 6 months in tap water (4), but were 224 pounds stronger than untreated samples (5) stored in 10 per cent alkali solution for the same length of time.

From these results it was concluded that water-gas tar if thoroughly incorporated in concrete offers, for a considerable period at least, an effective protection against the action of saline solutions.

TABLE 2.—*Strength of tar-treated and untreated mortar after one and six months in water and alkali solutions*

Sample No.	Treatment	Storage solution	Tensile strength		Compressive strength	
			28 days	182 days	28 days	182 days
			Lbs. per sq. in.	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
1	Untreated	Tap water	412	360	1,925	2,540
			435	362	1,995	2,480
			397	362	1,575	2,830
	Average		415	361	1,852	2,627
2	Untreated	3 per cent alkali	230	Disintegrated.	1,908	Disintegrated.
			220		2,300	
			235		2,230	
	Average		228		2,143	
3	6 days in water-gas tar.	3 per cent alkali	390	592	2,610	2,312
			495	490	2,340	2,432
			568	568	2,462	3,000
	Average		484	550	2,471	2,581
4	Untreated	Tap water	463	402		
			400	418		
			406	396		
			442			
	Average		427	405		
5	Untreated	10 per cent alkali	480	184		
			390	135		
			417	70		
	Average		429	130		
6	6 days in water-gas tar.	10 per cent alkali	515	363		
			502	330		
			465	370		
			463			
	Average		486	354		

TABLE 3.—*Action of alkali solution on concrete treated with tar and paraffin*

Sample No.	Treatment	Mix	Weight of samples	Loss of CaO in grams by months											
				1	2	3	4	5	6	7	8	9	10	11	12
			Kilograms												
1	Untreated	1:1½:3	10.684	7.7	9.2	9.2	9.2	9.4	9.4	10.4	11.2	12.2	12.2		
2	do	1:2:4	10.688	10.8	11.6	12.8	13.1		13.9	13.9	13.9	13.9	13.9	14.5	14.5
3	do	1:3:6	10.504	7.9	9.6	10.6	10.9	11.5	11.6	12.2	12.9	13.7	13.7		
4	6 coats of water-gas tar	1:1½:3	10.640	3.8	4.8	5.5	5.9	6.3	7.5	8.0	8.6	9.2	9.2		
5	do	1:2:4	10.872	7.5	8.6	10.3	10.3		11.0	11.6	11.6	11.8	11.8	12.8	12.8
6	do	1:3:6	10.504	5.6	6.8	7.5	7.8	7.8	7.8	9.1	9.4	9.8	9.8		
7	10 coats of water-gas tar	1:1½:3	11.120	4.3	5.7	6.2	6.7	6.7	7.1	7.1	7.7	7.7	7.7		
8	do	1:2:4	11.084	3.4	3.4	3.4	3.4		4.1	4.3	4.3	4.5	4.5	4.5	4.5
9	do	1:3:6	10.852	3.2	3.9	4.9	4.9	5.2	5.2	5.7	5.7	6.7	6.7		
10	10 coats of water-gas tar 1 coat of coal-gas tar	1:1½:3	11.324	0.3	1.4	1.1	0.4	0.4	1.1	1.4	1.4	1.6	1.8		
11	do	1:2:4	11.052	.0	.0	0.2	0.3		0.3	0.3	0.3	0.7	0.7	0.7	0.7
12	do	1:3:6	10.888	0.2	0.7	1.4	2.1	1.9	1.8	1.8	2.0	2.0	2.7		
14	24 hours in paraffin-kerosene	1:1½:3	11.853		5.0	5.9	5.9	6.0	6.4	6.4	6.4				
15	24 hours in paraffin-kerosene, 4 coats paraffin	1:1½:3	11.411	0.6	0.6	0.7	1.1	1.1	1.1						
16	do	1:2:4	11.596	.0	0.6	0.8	1.1	1.1							
17	do	1:3:6	11.416	1.2	1.2	1.2	1.2								

Sample No.	Treatment	Mix	Weight of samples	Gain in weight (grams) by months											
				1	2	3	4	5	6	7	8	9	10	11	12
			Kilograms												
1	Untreated	1:1½:3	10.684	304	300	340	340	332	332	332	336	336	348		
2	do	1:2:4	10.688	400	416	404	428		444	440	436	421	428	432	416
3	do	1:3:6	10.504	424	416	452	452	476	472	440	464	436	444		
4	6 coats of water-gas tar	1:1½:3	10.640	176	156	180	180	172	172	180	180	186	184		
5	do	1:2:4	10.872	276	284	260	288		300	292	292	288	288	288	276
6	do	1:3:6	10.504	288	296	324	324	336	332	328	332	320	320		
7	10 coats of water-gas tar	1:1½:3	11.120	112	120	148	148	140	140	148	148	148	156		
8	do	1:2:4	11.084	148	164	160	184		192	192	188	184	184	184	176
9	do	1:3:6	10.852	172	184	212	212	232	236	228	244	232	232		
10	10 coats of water-gas tar, 1 coat of coal-gas tar	1:1½:3	11.324	80	84	96	96	88	88	92	100	88	92		
11	do	1:2:4	11.052	132	136	124	126		144	136	132	128	132	124	116
12	do	1:3:6	10.888	124	126	160	160	172	176	168	180	172	172		
14	24 hours in paraffin-kerosene	1:1½:3	11.853	64	100	108	104	100	108	104	108				
15	24 hours in paraffin-kerosene, 4 coats of paraffin	1:1½:3	11.411	56	64	60	68	72	72						
16	do	1:2:4	11.596	104	112	128	124	124							
17	do	1:3:6	11.416	152	180	176	188								

SECOND SERIES OF TESTS

These preliminary investigations of the treatment of concrete with water-gas tar as a protection against alkali attack gave such encouraging results that the work was continued on a somewhat larger scale with coal-gas tar and paraffin in addition to water-gas tar as waterproofing materials.

The test pieces consisted of 4 by 6 inch cylinders made up of Portland cement, Potomac River sand, and gravel in the proportions of 1:1½:3, 1:2:4, and 1:3:6, which, after curing for 28 days in damp sand, were allowed to dry out for at least two weeks before being treated.

Four cylinders of each batch were treated as follows: One was painted with 6 coats of crude water-gas tar; one was painted with 10 coats of crude water-gas tar; one was painted with 10 coats of crude water-gas tar followed by 1 coat of coal-gas tar thinned with benzol; and one was immersed for 24 hours in a 20 per cent solution of paraffin and kerosene after which it was painted with 4 coats of the same solution. Water-gas and coal-gas tars were applied at normal room temperature; for paraffin treatment the concrete and solution were maintained at a temperature above 82° F.

After receiving the above treatment four cylinders of each batch, together with four untreated specimens, were stored in porcelain-lined, covered cans containing 1,600 cubic centimeters of a 3 per cent solution of sodium and magnesium suphate. At monthly intervals the specimens have been removed from the sulphate bath, allowed to dry out at room temperature for 48 hours, and weighed, and the gain in weight and loss in lime determined. This procedure will be continued until evidence of failure is noticeable in the untreated samples, when all cylinders will be tested for compressive strength and the broken material will be examined microscopically and chemically.

In Table 3 the results are recorded at monthly intervals extending over a period of 10 and 12 months' exposure to the action of the alkali. It will be observed that the effect of the tar treatment is similar for all concrete mixtures and that samples Nos. 10, 11, and 12 receiving 10 coats of water-gas tar and 1 coat of coal-gas tar indicate a minimum loss in lime and increase in weight. This is especially noticeable in the 1:2:4 mixtures (Nos. 2, 5, 8, and 11) where the loss in lime in 12 months is 14.5 grams for untreated samples (No. 2) and only 0.7 gram for samples treated with water-gas and coal-gas tars (No. 11), while the gain in weight, through absorbed moisture and secondary salts, is 416 grams for the untreated and only 116 grams for the sample treated with the two tars after 12 months' exposure to the alkali solution.

In regard to the paraffin treatment, it will be noted that samples from all batches receiving 4 coats of the paraffin solution after saturation (Nos. 15, 16, and 17) indicated about the same resistance to alkali attack as the tar-treated samples (Nos. 10, 11, and 12), whereas samples immersed for 24 hours in the paraffin bath but receiving no surface application of paraffin (No. 14) suffered an appreciable loss in lime after 8 months' exposure to the action of the alkali solution. The fact that gain in weight is not correspondingly large may indicate that the action which is taking place is confined largely to the surface of the specimens.

EFFECT OF ALKALI ON STRENGTH OF MORTAR

By CHAS. E. PROUDLEY, Junior Assistant Testing Engineer, U. S. Bureau of Public Roads

TESTS of mortar briquets made in the laboratory of the United States Bureau of Public Roads to determine to what extent the presence of alkali in mixing water is harmful indicate that—

1. The presence of alkaline sulphates in the mixing water in quantities of less than 1½ per cent has no serious effect on the strength or other physical properties of Portland cement mortar.

2. One per cent alkaline solutions in contact with cement mortar in mixes of 1:2 and leaner cause progressive decrease in the strength of the mortar.

3. The decrease in strength of mortar subjected to alkali action may be somewhat accelerated by the presence of considerable quantities of alkali in the mixing water, and for this reason the use of pure water in mixing is advisable under such conditions.

The specimens used in the tests were mortar briquets made with Ottawa sand and a good commercial brand of Portland cement. The alkali used had the following analysis:

	Per cent.
$CaCl_2$ (calcium chloride)	4. 50
$MgCl_2$ (magnesium chloride)	11. 20
$MgSO_4$ (magnesium sulphate)	3. 90
Na_2SO_4 (sodium sulphate)	80. 40
	100. 00

This represents the average analysis of a number of alkaline waters in which sulphates are the principal salts.

Figure 1 shows strength at various ages of specimens made with mixing water containing various percentages of alkali up to 5 per cent. The specimens were stored in water in the usual manner after the initial 24 hours in the moist closet. In a period of one year the use of the alkaline water has no objectionable effect; in fact, the specimens in which it is used have a slightly higher strength than the standard briquets.

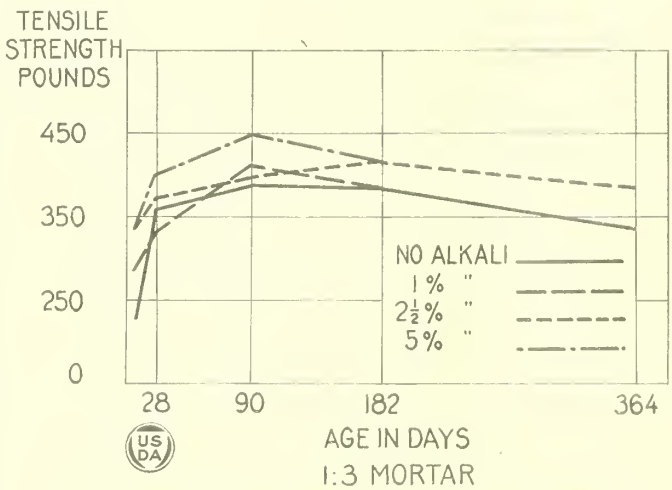


FIG. 1.—Effect of alkaline mixing water on tensile strength of mortar

It is noticeable that more than 2½ per cent alkali in the water does not have a correspondingly greater effect upon the strength and for this reason later tests were made using less alkali. Furthermore, analyses[3] of water from rivers, lakes, springs, and wells from every part of the country show that less than 1 per cent of the waters examined contain more than 1.5 per cent alkaline salts by weight. In fact, comparatively few contain more than 0.5 per cent of dissolved solids.

The results of the tests to determine the effect of alkali in contact with the specimens are shown in Figure 2. The curves shown at the top of the figure represent specimens stored under usual conditions. Below these are the curves of specimens cured in soil

[3] U. S. Geological Survey, Water Supply Paper 364.

(Continued on page 27)

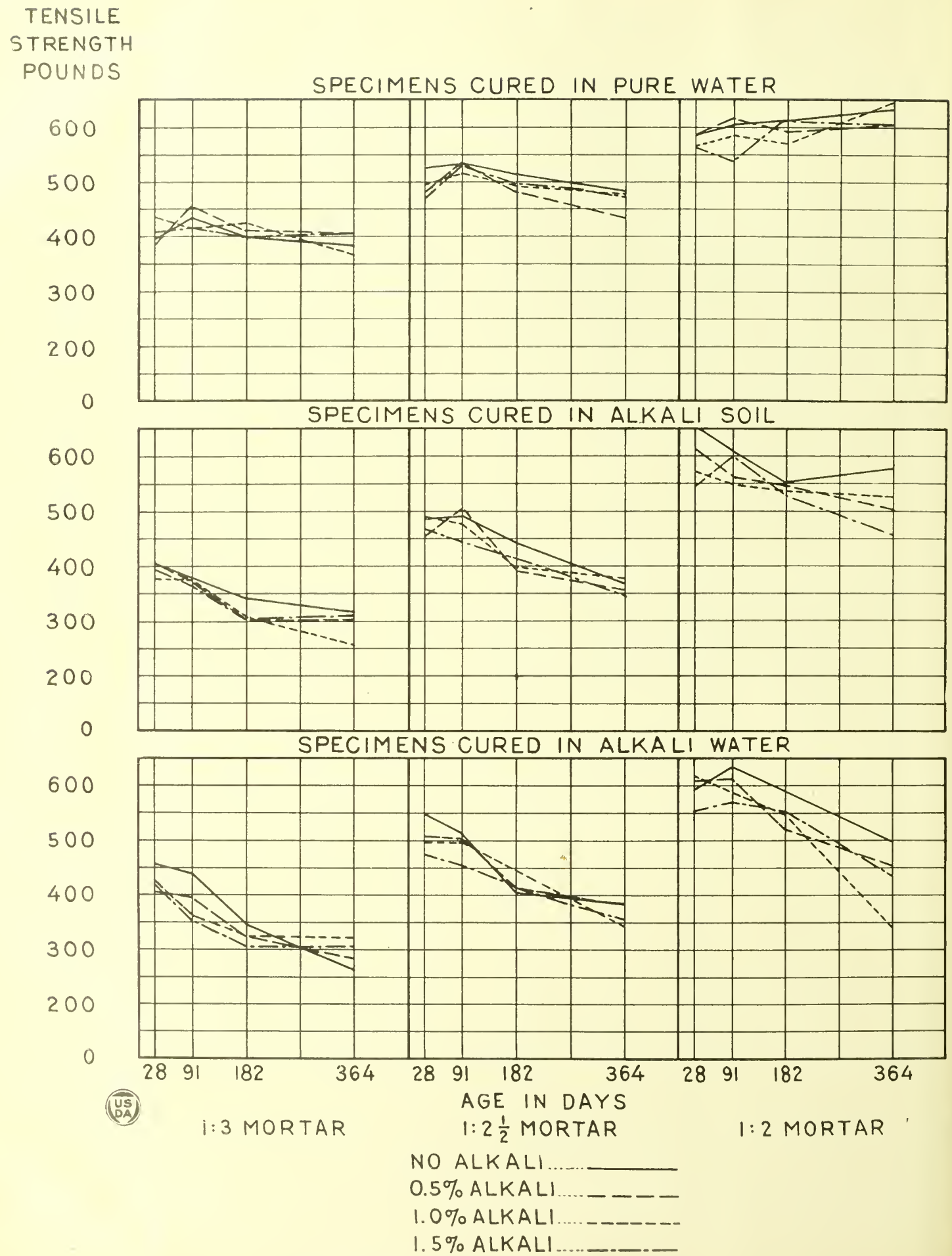


FIG. 2.—Effect of alkali in contact with specimens on their tensile strength

ROAD MATERIAL TESTS AND INSPECTION NEWS

UNIFORMITY OF CONCRETE

Information gathered recently in a number of States strikingly demonstrates the nonuniformity of concrete used in pavements. The information consisted of the results of compression tests on cores drilled from pavements and it can be safely stated that the situation disclosed is reflected in general by concrete pavements throughout the country.

For example, in one of the Eastern States the extreme range of strengths varied from 34 to 174 per cent of the average strength. This occurred with drilled cores which were five months old at the time of test. The minimum range for any particular age was from 86 to 121 per cent of the average strength. One of the Middle Western States reported on several projects an average strength of 4,417 pounds per square inch with an average minimum of 2,965 and an average maximum of 6,161 pounds per square inch.

Reflecting on the significance of this situation it may be pertinent to inquire which would be preferred, a pavement which has a range in strength from 2,000 to 4,000 pounds per square inch or one which has a range, say, from 2,700 to 3,300 pounds per square inch? The answer to this question is largely one of design. In the former case it is not practicable to take advantage of the higher or average strengths because there are parts of the pavement which yield only 2,000-pound concrete. With the more nearly constant strength, the design can be based with some degree of confidence on 3,000-pound concrete.

In other words, it is desirable from both economic and practical standpoints that the construction of concrete pavements be so controlled that the properties of the concrete will be uniform. If this uniformity is obtained, the design can be based on concrete of predetermined strength and the factor of safety required to cover the variations in the quality of the concrete can be materially reduced.

It is chiefly with the hope of controlling the construction in such a manner as to give this much-desired uniformity of the concrete that the United States Bureau of Public Roads proposes to try out an improved method of control on an actual paving project.

COMPRESSION TESTS OF CEMENT

The Bureau of Public Roads has begun a series of tests in cooperation with committee C1 on cement, of the American Society for Testing Materials for the purpose of studying further the possibility of substituting a compression test for the standard tension test of cement. The study will include comparative tests using 2-inch by 4-inch cylinders, 2-inch by 2-inch cylinders and 2-inch cubes in addition to the regulation tension briquettes as well as tests in which consistencies other than normal are employed. A novel feature of the work lies in the fact that the flow table will be used for measuring the consistency of the mortar. From the results it is hoped to determine whether or not the flow table may be used for the determination of consistency in routine testing. Full details including working drawings of the flow table will be sent to anyone interested on request.

COOPERATIVE TESTS OF PAVING BRICK

A study of the standard rattler test for paving brick has been undertaken by the Bureau of Public Roads in an effort to determine the effect of size of brick on the percentage of loss. The work is being done at the request of and in cooperation with the Committee on Brick of the American Society for Testing Materials. Other cooperating agencies are the Rensselaer Polytechnic Institute, Troy, N. Y., and the Department of Public Works of the city of Buffalo. Testing engineers have realized for some time that the so-called 3-inch brick may be somewhat penalized when tested under the specifications as to minimum allowable rattler loss which are used for the larger sizes. Although a charge of 3-inch brick weighs considerably less than a corresponding charge of 4-inch brick, the total length of edge exposed to wear is not proportionately smaller. Due to the fact that the loss is computed on the basis of percentage of original weight and also to the fact that by far the greatest amount of wear in the test comes on the edges of the brick, it will be seen that quite an error may possibly be introduced. Many specifications have recognized this fact by providing for a differential of 1 or 3 per cent loss in the case of the smaller size. These figures are, however, for the most part arbitrary and not based on test data. The tests which are to be made by these three laboratories should throw some light on the question as to just what this differential should be.

EFFECT OF ALKALI ON STRENGTH OF MORTAR

(Continued from page 25)

saturated with a saturated solution of alkali and the lower set is for specimens stored in a 1 per cent solution of alkali.

Referring to the curves in Figure 2, it will be noted that alkali in the mixing water in amounts of 1½ per cent or less has an immaterial effect upon the strength, although a tendency toward slightly lower strength in this series may be noticed. This may be due to a number of causes such as a slightly lower density resulting from the presence of the alkaline salts in some chemical combination with the cement or some of its constituents, or lower cementing qualities due to this chemical action alone. Where the specimens have been stored in alkali, it seems that the presence of alkali in the mixing water assists disintegration perceptibly. Inasmuch as it would seldom be necessary to use alkaline mixing water unless the concrete would later be subjected to contact with alkali, this point is worthy of note.

Time of set and soundness tests on cement pats mixed with alkaline waters of various strengths show that neither time of set nor soundness is altered to any appreciable extent by the presence of alkali in the mixing water.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

APRIL 30.1924

STATES	FISCAL YEARS 1917-1923			FISCAL YEAR 1924									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1923			PROJECTS COMPLETED SINCE JUNE 30, 1923			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 3,789,011.74	$ 1,794,703.21	367.9	$ 528,287.87	$ 257,160.91	66.2	$ 13,920,666.31	$ 6,949,200.70	853.4	$ 652,035.75	$ 278,708.11	4.4	$ 431,137.51	Alabama
Arizona	6,618,025.52	3,196,689.68	372.1	1,194,453.62	727,736.97	104.8	2,262,520.86	1,323,211.35	173.4	825,736.07	504,607.28	45.0	691,452.16	Arizona
Arkansas	6,623,206.72	3,308,572.12	736.2	2,316,145.05	1,045,641.62	132.3	4,256,316.15	1,732,591.09	223.7	3,515,723.24	845,234.70	215.6	871,603.40	Arkansas
California	6,495,547.60	3,538,967.90	340.5	4,145,805.72	1,962,804.58	187.0	11,613,685.98	6,111,198.17	414.7	540,834.58	379,356.16	25.3	2,635,958.42	California
Colorado	6,470,695.66	3,136,236.67	426.6	1,189,833.78	651,192.83	68.7	4,400,763.35	2,398,531.33	169.7	250,933.76	140,822.34	11.5	1,871,613.77	Colorado
Connecticut	2,566,643.95	1,026,148.85	60.7	126,610.43	64,305.21	3.8	2,655,321.05	986,119.24	49.2	755,761.37	222,495.20	12.2	606,612.69	Connecticut
Delaware	2,196,610.22	621,154.83	46.2	716,598.97	317,815.00	22.7	988,468.80	389,767.50	27.5	76,668.90	31,961.10	0.2	13,206.40	Delaware
Florida	69,466.31	29,700.63	15.6	504,477.62	288,175.17	23.3	8,578,162.86	4,232,785.06	265.5	1,037,874.30	440,221.40	13.7	408,668.22	Florida
Georgia	13,960,499.47	6,363,703.60	925.5	2,966,428.24	1,472,330.06	274.9	7,865,396.47	3,809,568.64	672.1	1,537,997.01	692,146.45	102.8	129,065.26	Georgia
Idaho	6,660,673.73	3,272,111.01	426.1	1,141,088.96	727,622.09	74.2	1,404,050.80	826,531.30	96.2	727,191.21	408,930.73	64.0	605,918.86	Idaho
Illinois	25,211,526.23	11,414,291.96	743.6	768,612.90	379,214.16	29.2	14,394,340.81	7,150,290.36	485.9	1,250,966.78	616,070.33	36.9	662,757.21	Illinois
Indiana	5,513,487.76	2,668,277.66	157.0	1,945,251.03	928,993.26	64.6	12,486,232.40	6,121,359.53	411.3	2,109,233.76	1,032,191.18	81.4	1,621,667.16	Indiana
Iowa	16,040,079.62	6,056,136.48	1,059.4	6,952,470.02	3,097,340.68	600.5	6,061,281.68	2,832,857.08	649.8	2,703,671.60	1,228,200.00	101.7	43,354.63	Iowa
Kansas	12,216,426.90	4,246,436.34	349.7	4,392,131.67	1,602,266.68	129.9	16,048,072.99	5,925,834.08	648.1	2,511,965.23	1,058,678.20	115.1	382,841.66	Kansas
Kentucky	6,620,289.80	2,789,991.16	266.0	4,061,627.63	1,768,710.23	146.9	7,316,679.52	3,535,492.32	322.1	471,726.68	235,863.33	19.0	630,097.62	Kentucky
Louisiana	7,131,394.90	3,157,128.03	690.4	1,116,208.62	367,989.39	54.2	5,682,810.05	2,729,308.82	376.8	16,405.36	8,202.68	0.2	7,511.49	Louisiana
Maine	4,662,854.19	2,242,963.03	159.8	2,248,204.69	1,066,972.35	70.9	907,304.26	451,804.19	34.8	713,577.40	354,204.36	34.0	297,674.71	Maine
Maryland	6,309,394.94	2,624,843.64	186.0	1,427,449.76	676,878.38	57.3	1,601,912.79	669,855.76	62.0	234,966.08	10,000.00	6.2	131,427.32	Maryland
Massachusetts	6,841,975.16	2,602,957.68	163.7	1,967,316.05	767,062.11	40.7	5,120,267.44	1,849,601.03	96.5	782,886.11	257,689.00	12.0	1,132,763.76	Massachusetts
Michigan	9,528,041.53	4,343,817.14	363.4	3,051,384.48	1,305,637.89	106.5	10,448,363.61	6,102,961.38	402.9	3,200,506.31	1,554,684.84	84.7	1,345,846.02	Michigan
Minnesota	19,826,671.37	8,004,277.87	1,893.4	4,208,869.87	1,881,565.20	398.6	7,366,883.13	3,020,954.43	626.8	439,512.73	206,000.00	66.0	85,714.94	Minnesota
Mississippi	4,746,771.31	2,264,557.86	462.4	2,570,568.66	1,260,281.77	143.4	8,352,228.83	4,168,183.90	532.7	399,842.08	176,586.34	16.0	347,291.48	Mississippi
Missouri	5,272,767.08	2,358,848.60	365.6	5,662,471.34	2,673,920.07	383.6	14,148,945.38	6,813,519.37	618.1	1,772,165.49	821,039.81	103.3	2,849,374.40	Missouri
Montana	7,935,185.30	3,696,053.50	711.6	803,232.04	420,005.19	62.0	1,832,052.67	1,267,543.63	194.1	1,236,344.25	702,460.16	140.6	3,116,871.33	Montana
Nebraska	3,159,614.47	1,449,675.04	476.6	4,210,786.91	2,014,231.26	858.5	5,864,159.94	2,824,263.01	661.1	1,951,094.47	970,367.24	182.7	2,615,254.11	Nebraska
Nevada	3,049,109.29	1,562,111.35	203.6	411,136.23	291,613.63	22.0	3,994,660.18	3,436,013.46	343.5	446,060.81	341,009.43	10.7	312,049.88	Nevada
New Hampshire	2,336,997.00	1,129,004.69	140.5	739,753.19	358,862.89	30.8	691,313.98	279,768.77	21.5	266,029.38	119,644.07	7.4	182,068.57	New Hampshire
New Jersey	5,423,362.96	1,975,783.62	112.1	1,794,735.66	562,627.10	29.4	8,552,430.81	2,153,320.04	49.0	1,669,699.11	538,823.99	19.1	422,379.32	New Jersey
New Mexico	4,706,909.12	2,433,663.68	617.1	599,377.33	325,186.00	97.2	4,896,222.57	3,100,904.09	583.4	1,600,613.04	1,079,939.15	104.8	464,110.20	New Mexico
New York	9,626,633.82	4,356,659.70	290.6	6,978,203.72	2,944,367.63	222.8	22,547,626.96	8,929,161.47	526.1	5,506,000.00	2,006,425.00	119.3	4,807,428.44	New York
North Carolina	11,770,363.76	5,325,763.37	826.6	797,369.21	350,994.29	57.9	8,864,391.81	3,148,515.67	237.7	2,512,967.10	1,154,324.36	68.6	617,407.16	North Carolina
North Dakota	5,142,157.65	2,492,599.09	792.6	3,660,544.68	1,812,626.17	770.0	2,127,951.19	1,026,483.84	334.7	853,070.62	426,535.21	140.0	1,426,703.66	North Dakota
Ohio	22,840,050.09	7,956,326.17	662.8	9,618,128.44	3,686,542.82	281.0	11,612,728.70	4,400,022.84	315.7	2,549,123.62	862,626.06	80.5	428,842.43	Ohio
Oklahoma	9,795,634.54	4,417,768.46	371.5	3,130,150.61	1,440,648.45	124.3	7,822,145.92	3,787,990.79	352.1	1,571,115.66	733,123.45	74.6	403,962.14	Oklahoma
Oregon	10,391,362.48	4,762,736.38	634.1	1,691,610.69	1,036,357.41	121.5	2,662,216.36	1,416,202.87	144.0	138,208.96	65,675.96	17.5	28,356.23	Oregon
Pennsylvania	33,090,029.45	12,615,068.79	652.8	3,225,176.88	1,328,845.47	68.3	7,181,910.81	2,424,402.50	126.3	11,829,122.36	3,381,970.00	216.8	1,485,383.03	Pennsylvania
Rhode Island	1,464,076.36	647,634.96	36.6	290,321.90	131,693.00	7.4	1,087,787.06	411,759.49	22.7				376,428.68	Rhode Island
South Carolina	6,181,485.41	2,865,754.37	655.1	1,626,786.06	739,184.34	247.7	6,685,566.58	2,640,586.98	503.6	647,463.82	203,684.76	46.0	164,307.39	South Carolina
South Dakota	4,758,907.24	2,320,007.33	634.6	3,579,964.08	1,767,665.78	432.6	5,221,804.03	2,681,558.80	645.9	1,066,475.00	607,154.06	110.3	233,149.86	South Dakota
Tennessee	3,210,970.94	1,535,762.16	112.5	2,064,853.17	1,019,402.48	87.6	13,827,431.86	6,724,982.26	447.6	2,099,483.67	800,842.73	62.5	314,906.38	Tennessee
Texas	29,172,501.06	11,094,069.77	2,259.6	11,347,463.61	4,293,943.98	767.3	24,943,423.31	9,941,857.22	1,648.0	4,252,173.84	1,667,637.19	251.0	316,636.08	Texas
Utah	2,175,007.41	1,196,437.58	116.6	1,030,357.45	627,434.33	98.9	3,399,600.07	2,101,299.95	227.3	860,682.07	635,336.20	51.4	708,223.94	Utah
Vermont	1,370,716.63	669,433.76	53.1	651,398.63	273,335.37	21.3	1,144,973.83	651,274.77	34.6	363,118.12	173,388.64	8.9	500,921.20	Vermont
Virginia	6,823,978.36	3,279,029.66	402.6	2,690,547.72	1,262,704.51	134.7	8,850,281.55	4,078,223.14	325.3	661,161.37	130,773.51	18.3	393,659.73	Virginia
Washington	10,356,980.82	4,819,105.70	419.8	618,239.13	273,551.37	36.3	2,354,076.03	1,197,800.00	97.1	1,069,358.60	474,900.00	36.2	98,012.76	Washington
West Virginia	4,100,226.73	1,800,932.41	206.3	1,389,522.22	564,109.12	49.3	4,740,449.70	2,084,116.50	166.8	574,610.20	260,790.10	25.1	245,908.40	West Virginia
Wisconsin	15,504,691.39	5,996,969.24	1,077.6	2,671,162.12	1,177,011.47	208.9	3,204,764.79	1,526,918.42	150.2	109,324.13	64,662.06	9.3	3,045,269.49	Wisconsin
Wyoming	4,601,439.36	2,210,774.20	534.8	1,187,011.02	669,291.38	123.6	4,078,154.44	2,531,599.30	272.9	417,005.83	168,594.52	48.8	150,719.47	Wyoming
TOTALS	407,704,641.14	174,044,673.82	23,297.2	122,156,059.06	54,693,411.86	8,194.4	335,968,620.56	153,738,096.42	16,215.4	70,656,176.51	28,962,510.36	3,011.5	40,561,307.55	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 49,154,309.22 Federal aid $ 21,750,040.45 Miles 2,289.9

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

REPORTS

Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.
Report of the Chief of the Bureau of Public Roads for 1920.
Report of the Chief of the Bureau of Public Roads for 1921.
*Report of the Chief of the Bureau of Public Roads for 1922. 5c.
*Report of the Chief of the Bureau of Public Roads for 1923. 5c.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
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*389. Public Road Mileage and Revenues in the Central, Mountain, and Pacific States, 1914. 15c.
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FARMERS' BULLETINS

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*505. Benefits of Improved Roads. 5c.
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SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
74. State Highway Mileage and Expenditures for the Calendar Year 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.
Public Roads Vol. III, No. 25. Automobile Registrations, Licenses, and Revenues in the United States, 1919.
Vol. III, No. 29. State Highway mileage, 1919.
Vol. III, No. 36. Automobile Registrations, Licenses, and Revenues in the United States, 1920.
Vol. IV, No. 5. Automobile Registrations, January 1 to July 1, 1921.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

THE FEDERAL AID HIGHWAY SYSTEM

THE Federal highway act, approved November 9, 1921, provided for the selection by the State highway departments of a system of highways not to exceed 7 per cent of the total highway mileage of each State. Upon this system, all apportionments of Federal aid are to be expended. The system is to be divided into two classes, primary and secondary, of which the former are not to exceed three-sevenths of the total, the remainder to be of the latter class. The Secretary of Agriculture was given authority to approve in whole or in part the systems as designated by the State highway departments or to require modifications or revisions thereof.

THE total mileage of existing highways certified by the States was 2,866,061 miles. The States designated by maps and route description, systems of main roads totaling in mileage not more than 7 per cent of the certified mileage. The systems for groups of adjoining States were reviewed by representatives of the States and of the Bureau of Public Roads meeting in a series of conferences for the principal purpose of connecting the systems at State lines. In this way the entire system was coordinated and recommended to the Secretary of Agriculture by the Bureau of Public Roads. The system as approved by the Secretary of Agriculture and represented by the map published November 1, 1923, includes 168,881 miles which is 5.9 per cent of the certified mileage.

UP TO March 1 the Federal-aid highways which had been completed since the passage of the Federal-aid road act in 1916 totaled 33,036 miles, and 13,800 miles were under construction and reported as 59 per cent complete. The total of roads completed and under construction amounted therefore to 46,836 miles. Of the mileage reported as completed on February 29, 6,307 miles had been completed during the current fiscal year. All but a very small percentage of this mileage is on the Federal-aid highway system as now established.

IN ADDITION to the roads of the system improved with Federal aid, parts of it have been improved without Federal assistance. A careful study is being made of the improvement status of the system and an approximate estimate based upon these incomplete studies is that at the end of the year there were about 60,000 miles of surfaced roads and 8,700 miles graded, which leaves nearly 110,000 miles yet to be surfaced.

TO BRING this system up to serviceable standards, therefore, within the full decade ahead, will mean a surfacing program of about 11,000 miles for each of the 10 years; this in addition to the additions to the system, the separation of grade crossings, reconstruction, and much other work necessary.

A22.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 4 ▽ JUNE, 1924

THE OLD WAY TOLL - AND THE NEW

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

VOL. 5, NO. 4 | JUNE, 1924 | H. S. FAIRBANK, Editor

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THE INCIDENCE OF THE HIGHWAY TAX BURDEN

By HENRY R. TRUMBOWER, Economist, U. S. Bureau of Public Roads

THE TOTAL income for all rural highway purposes in 1921 was $1,149,437,896, according to a survey of rural highway mileage, funds and expenditures in each of the 48 States made by the United States Bureau of Public Roads. This amounts to a per capita contribution on the part of the people of the United States of $10.90, based upon the population figures of the 1920 census. The highway income was derived from a number of sources and can not be considered as having been levied as a uniform burden upon the taxpayers and public in general.

That there is a marked tendency to collect a larger proportion of highway funds from the owners and users of motor vehicles is noted in analyzing the various sources of these highway funds and comparing them with similar sources in previous years. A survey of highway revenues and expenditures made in 1914 showed that, out of a total highway income of $240,262,784, the collections from motor vehicles amounted to $12,382,031, or 5.1 per cent of the total. In 1921, seven years later, the motor vehicle owners and operators paid $118,942,706 in motor-vehicle fees and $3,685,460 in gasoline taxes, a total of $122,626,166, or 10.6 per cent of the total income for highway purposes. A similar compilation of the payments made by owners and operators of motor vehicles for licenses and permits in 1923 shows a total contribution on their part of $188,970,992. In the same year the taxes upon the sale of gasoline levied by 35 States amounted to $36,813,939. The motor vehicles, accordingly, contributed a total of $225,784,931.

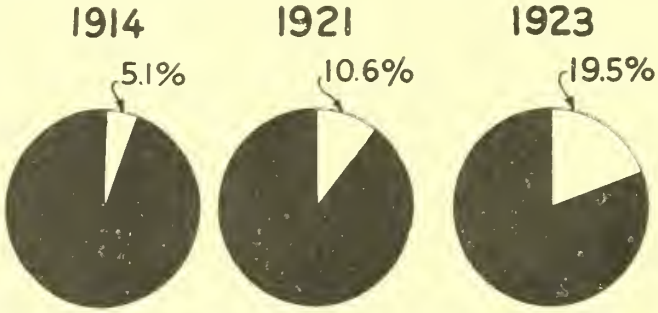
It is estimated that the highway income and expenditures for 1923, based upon facts and information thus far obtained, were substantially the same as for 1921. It follows, therefore, that in 1923 the contribution of the motor vehicle towards the total highway income was about 19.5 per cent. This shows a very substantial increase in the relationship of motor-vehicle revenues to the total highway income, as compared with 1914, when it was only 5.1 per cent. Between 1921 and 1923, it is observed, the ratio almost doubled. This increase in the contribution made by the motor vehicle is accounted for by the increase in the total number of vehicles registered and in the payment per vehicle, which rose from $11.70 in 1921 to $15 per vehicle in 1923.

Another very substantial part of the total highway income was derived from the sale of bonds, both State and local; $438,109,273, or 38.1 per cent, was obtained in this manner. This amount, which represents a deferred payment of $4.15 per capita, includes all receipts from the sale of highway bonds during 1921 and all cash from previous bond sales on hand at the beginning of the year. At the end of the year the total amount of State and local bonds outstanding was $1,222,312,300, a per capita issue of $11.50, or slightly more than the total of a single year's income. The interest on this amount and the requisite provision for retirement constitute a current obligation which must be paid in addition to the costs of highway construction and maintenance from the highway revenues. The Census Bureau reports at the end of 1922 a total indebtedness of $8,695,906,000 for the country as a whole, including State, county, local, and municipal indebtedness. The highway bonds amount to about 14 per cent of this total indebtedness.

The income for all rural road purposes includes also payments to the different States by the Federal Government amounting to $79,333,226 made in accordance with the provisions of the Federal-aid act, a contribution on the part of the Government which amounts to an assessment of approximately 75 cents per capita.

LESS THAN ONE-HALF OF HIGHWAY INCOME RAISED BY PROPERTY TAXES

General property taxes contributed $415,680,010 to the year's total highway income. In addition, $93,689,221 was derived from other sources, a large part of which amount consisted of appropriations from funds in the respective State treasuries, which in turn had their sources in tax levies. For our present purposes we may consider both of these items as having been



Ratio of motor-vehicle revenues to total highway income in 1914, 1921, and 1923

payments made by the general public and may therefore be regarded as taxes although a small portion was derived from other sources which could not be clearly segregated. The sum of these two items, $509,369,231 or 44.4 per cent of the total highway income is regarded in this analysis as the direct contribution of the public towards the cost of highway construction and maintenance and towards the principal and interest payments of the bonds outstanding. The per capita tax burden for highway purposes is, therefore, $4.83. The total highway income for 1921 is divided and allocated as follows:

Total highway income, 1921

Source	Amount	Per capita
Motor vehicles	$122,626,166	$1.17
Bonds	438,109,273	4.15
Federal aid	79,333,226	.75
General property taxes and other sources	509,369,231	4.83
Total	1,149,437,896	10.90

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The Census Bureau in a recent publication reports that in 1922 the revenues of all the States for governmental purposes were $4,224,541,865. This includes general property taxes, special taxes, poll taxes, licenses, permits, and special assessments collected by the States and by all of their political subdivisions. It is not unfair to assume that the public revenues for 1921 were approximately the same as those reported for the subsequent year and the following comparison is made on that basis. In deducting from the year's total public revenues, $122,626,166 collected as motor-vehicle license fees and gasoline taxes, there is left $4,101,915,699 as the amount received from other sources or a contribution for governmental purposes of $38.80 per capita. Of this total amount, exclusive of motor-vehicle revenues, $509,369,231 was collected from the public for highway purposes of $4.83 per capita. It follows, therefore, that of every dollar collected by the States, counties, cities, townships, and other taxing districts only 12.4 cents was used for highway purposes.

ONE-QUARTER OF NEW ENGLAND ROAD BILL PAID BY VEHICLE OWNERS

On account of the wide variations which exist in different parts of the country as to density of population, road mileage, character of road construction, amount of highway traffic, etc., it is essential in order to make worth while comparisons to make an examination of the incidence of these highway costs in the different groups of States.[1] In the six states comprising the New England division the total highway income was $46,455,284 in 1921, or $6.25 per capita. Of this amount $11,629,091 or 25.1 per cent, consisted of license fees contributed by the motor vehicle. The revenues derived from the sale of bonds were $5,889,745 which constituted only 12.7 per cent of the total highway income; and amounted to only 80 cents per capita.

The amount received from the Federal Government as Federal aid was $2,904,636, or 6.2 per cent of the total. The general property taxes and revenues derived from other sources were $26,031,762; this constituted 56 per cent of the total highway income and amounted to $3.52 per capita. According to the report of the Census Bureau the States and their political subdivisions collected $374,743,288 in taxes, fees, licenses, etc. After deducting the motor vehicle license fees there is left $363,114,197 as the total public revenues derived from all other sources, or about $49.00 per capita, of which amount $3.52 was collected and used for rural highway purposes. This indicates that 7.2 cents of every dollar collected by State and local governments was devoted to highway purposes.

The total highway income of the three States comprising the Middle Atlantic division was $168,305,433, or $7.50 per capita. Of this amount $22,340,418, or 13.3 per cent, was obtained from motor-vehicle license fees. Bond sales furnished the source of $59,543,258 of the total, or 35.4 per cent; this was an obligation of $2.68 per capita for the year. The Federal Government contributed $7,441,515, or 4.4 per cent of the total amount. The general property taxes and the revenues derived from other sources were $78,980,242, which was 46.9 per cent of the total highway income and constituted a per capita burden of $3.52. The total amount of public revenues collected in these States by the State and local governments was $1,056,916,160, after making a deduction for the amount representing motor-vehicle license fees. The total per capita collections made for all purposes amounted to $47.30. As already indicated, of this amount $3.52 per capita was credited to highway funds, which means that out of every dollar collected from the public only 7.5 cents was used for highway purposes.

In the East North Central division the 1921 total highway income was $281,139,024, representing a per capita contribution of $13.10. The motor-vehicle fees amounted to $27,432,261, or 9.8 per cent of the total. From the sales of bonds these States received $101,550,318, which was 36.1 per cent of the total highway income and a per capita obligation of $4.70. The Federal aid received by these States amounted to $11,887,987, or 4.2 per cent of the total highway income. General property taxes destined for highway purposes and the revenues from other sources were $140,268,558, or 49.9 per cent of the total highway income. They constituted a per capita burden of $6.50. After making a deduction of the motor-vehicle fees, the total public revenues collected in these States amounted to $972,279,786, or $45.20 per capita. Out of this per capita tax collection, $6.50 was credited to the highway account, which indicates that 14.4 cents out of every dollar of public revenues was used for highway purposes.

In the year 1921 the total highway income of the West North Central division amounted to $153,642,716 or $12.25 per capita. The motor-vehicle fees constituted $20,574,538 and represented 13.4 per cent of the total highway income. These States derived $34,291,178, or 22.3 per cent, of the total from the sale of bonds, which represented a per capita obligation for the year of $2.75. The revenues derived from Federal aid were $14,636,169, or 9.5 per cent of the total. General property taxes and revenues from other sources for highway purposes constituted $84,140,831, or 54.8 per cent of the total highway income, which resulted in a per capita burden of $6.70. The total public revenues in this division of States were $539,197,660 after subtracting the fees received from motor vehicles. These total collections from the public represented a per capita burden of $43.00, $6.70 of which was collected for highway purposes, which means that out of every dollar collected 15.5 cents could be designated as highway income.

ONE-SIXTH OF ALL TAXES FOR HIGHWAYS IN SOUTH ATLANTIC STATES

The 1921 highway income for the South Atlantic division was $137,657,698, or $9.85 per capita. The revenues derived from motor vehicle fees and gasoline taxes were $11,860,998, or 8.6 per cent of the total. These States received 49 per cent of the total highway income, or $67,406,730 from the sales of bonds, which amounted to a per capita obligation of $4.80. Of the total highway income Federal aid constituted 8.2 per cent or $11,267,126. The general property taxes and receipts from other sources were $47,122,844, which was 34.2 per cent of the total highway income, or a collection of $3.40 per capita. After deducting the reve-

[1] The various groups of States referred to in the article are listed as follows:
New England division: Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.
Middle Atlantic division: New York, New Jersey, Pennsylvania.
East North Central division: Ohio, Indiana, Illinois, Michigan, Wisconsin.
West North Central division: Minnesota, Iowa, Missouri, North Dakota, South Dakota, Nebraska, Kansas.
South Atlantic division: Delaware, Maryland, Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida.
East South Central division: Kentucky, Tennessee, Alabama, Mississippi.
West South Central division: Arkansas, Louisiana, Oklahoma, Texas.
Mountain division: Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, Nevada.
Pacific division: Washington, Oregon, California.

nues derived from motor-vehicle fees and gasoline taxes these States collected from the public the sum of $283,145,170 or $20.10 per capita for all public purposes. Of this total income only $3.40 was devoted to highway purposes which means, however, that 16.8 cents out of every dollar collected from the public was credited to the highway account.

In the East South Central division the total highway income was $60,280,684 in 1921 which amounted to $6.80 per capita. The motor-vehicle fees and gasoline taxes, $5,108,387 represented 8.5 per cent of the total highway income. The revenues which accrued from the sales of bonds were $25,551,347, or 42.5 per cent; this was a capital obligation of $2.90 for that year. The States in this division received $5,281,475 as Federal aid which constituted 8.7 per cent of the total. General property taxes and collections from other sources supplied $24,339,475, 40.3 per cent of the total, or $2.70 per capita. In this division the total public revenues, exclusive of motor-vehicle fees and gasoline taxes, were $155,501,192, or $17.55 per capita. This total per capita tax collection included the $2.70 which went for highway purposes, which means that 15.3 cents out of every dollar collected were turned over to the highway fund.

The total highway income in the West South Central division was $138,504,160 which represented a per capita burden of $13.50. The motor-vehicle fees and gasoline taxes amounted to $7,496,965 and constituted 5.4 per cent of the total highway income. The receipts from the sale of bonds were $82,127,751, or 59.3 per cent, an obligation of $8 per capita. The aid derived from the Federal Government was $10,929,721, or 7.9 per cent of the total highway income. The general property taxes and the revenues derived from other sources amounted to $37,949,723 or 27.4 per cent of the total highway income, a per capital burden of $3.70. After making an allowance for the revenues derived from motor-vehicle fees and the gasoline taxes these States collected $235,959,540 or $23 per capita. Of this amount $3.70 was destined for highway purposes, or 16.1 cents of every dollar collected from the public.

The States of the Mountain division raised $61,367,959 in 1921 for highway purposes, which represents a per capita burden of $18.40. Seven per cent of the highway income, $4,305,524, was derived from motor-vehicle fees and gasoline taxes. The bond sales amounted to $19,908,036, or 32.4 per cent of the total highway income, a per capita obligation of $6. The Federal Government contributed toward highway improvements $9,731,542, which constituted 15.8 per cent of all highway income. General property taxes and revenues derived from other sources amounted to $27,422,857, constituting 44.8 per cent of the total highway income, a per capita burden of $8.20. Exclusive of motor-vehicle fees and gasoline taxes, these States received $162,761,525 in taxes and payments of all kinds from the public, or $48.90 per capita. Of this amount, $8.20 was used for highway purposes. Of every dollar collected from the public 16.8 cents were credited to the highway funds.

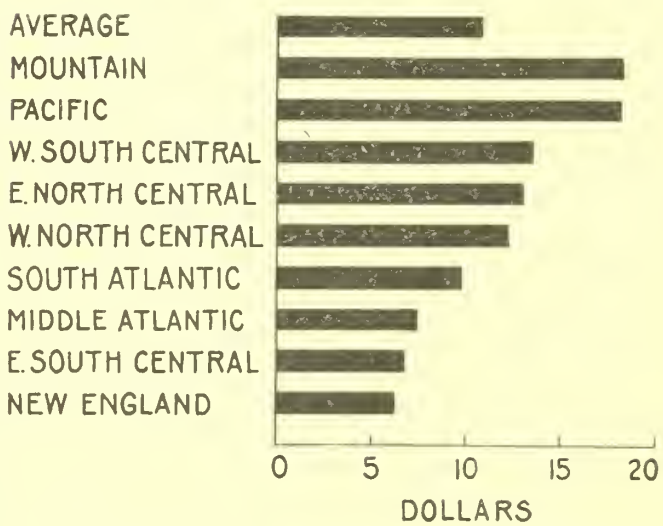
The total highway income in the Pacific division was $102,084,938 in 1921, amounting to $18.30 per capita. The motor-vehicle license fees and gasoline taxes were $11,877,984, or 11.6 per cent of the total. The revenues derived from bond sales were $41,840,910, or 41 per cent of the total highway income, a per capita obligation of $7.50. The Federal-aid receipts were $5,253,105, which constituted 5.1 per cent of the total highway income. The revenues derived from general property taxes and from other sources were $43,112,939, or 42.3 per cent of the total highway income, amounting to $7.75 per capita. In these States the total taxes, fees, etc., collected exclusive of motor-vehicle revenues were $333,040,929, or $59.20 per capita. Of this total amount collected from the public, $7.75 was devoted to highways and 13.1 cents out of every dollar collected were used for highway purposes.

WIDE VARIATIONS IN HIGHWAY INCOME OF GEOGRAPHIC GROUPS

As the per capita allocations present a good many variations in the several groups of States, it may be of interest to compare the several items by groups.

Total highway income

Divisions	Amount	Per capita
New England	$46,455,284	$6.25
Middle Atlantic	168,305,433	7.50
East North Central	281,139,024	13.10
West North Central	153,642,716	12.25
South Atlantic	137,657,698	9.85
East South Central	60,280,684	6.80
West South Central	138,504,160	13.50
Mountain	61,367,959	18.40
Pacific	102,084,938	18.30
Total	1,149,437,896	10.90



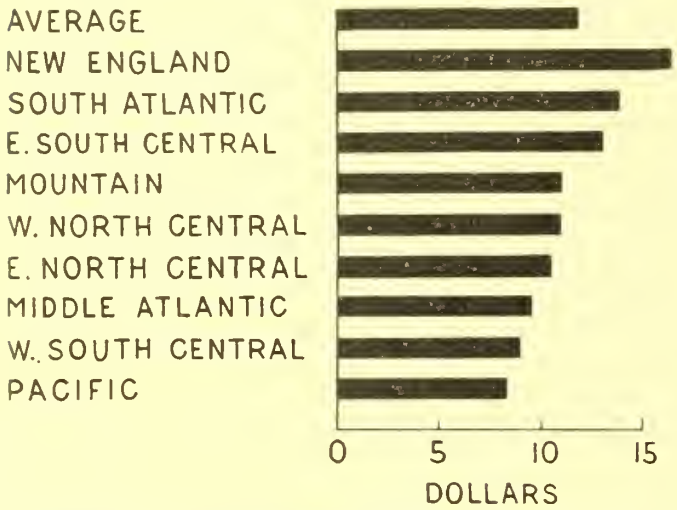
Total highway income per capita, 1921, by geographic divisions

The average per capita income for the whole country for rural highway purposes was $10.90. In four of the divisions, New England, Middle Atlantic, South Atlantic, and East South Central, the per capita highway income was less than the average for the country, and in the remaining divisions it was above the average. The per capita income of $6.25 in New England was the lowest found in any group and the $18.40 of the Mountain States was the highest, although the Pacific States, with $18.30, were a close second. In both of these groups it will be noted the per capita highway income was almost three times as large as in New England. This wide variation is explained by the fact that in these Western States the total income exceeded by very substantial amounts the total highway income of the New England States, while at the same time the population in the New England area was far in excess of the population of the

two western groups. In the Mountain States, where the per capita highway income was $18.40, the density of population based upon the 1920 census was only 3.9 persons per square mile. In the New England States, where the total highway income averaged only $6.25 per capita, the density of population was 120 persons per square mile, and in the Middle Atlantic States, with a population density of 222 persons per square mile, the per capita highway income was $7.50. In general it may be said that the per capita highway income tends to vary inversely with the density of population.

MOTOR-VEHICLE FEES AVERAGE NEARLY $12 PER VEHICLE

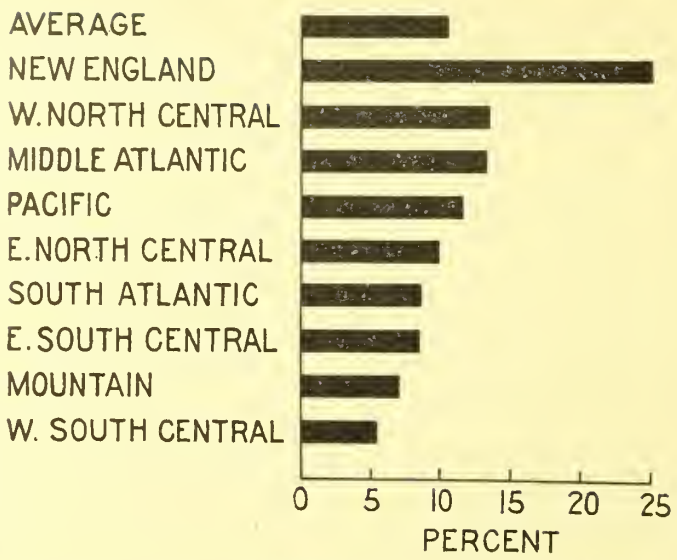
The motor-vehicle revenues consisted almost wholly of the fees charged for licenses issued for the operation of automobiles, motor trucks, and motor vehicles. Gasoline taxes are also included, but in 1921 these taxes amounted to only a small sum. The collection of these revenues exacted from motor-vehicle owners and users may be summarized as follows. The average amount paid by the owners is also shown.



Average highway income derived from motor vehicles, per motor vehicle, 1921, by geographic divisions

Division	Amount of motor-vehicle revenues credited to highway income, 1921	Number of motor vehicles	Average revenue per motor vehicle
New England	$11,629,091	706,312	$16.40
Middle Atlantic	168,305,433	1,774,614	9.55
East North Central	27,432,261	2,602,617	10.50
West North Central	20,574,538	1,871,157	10.95
South Atlantic	11,860,998	859,908	13.80
East South Central	5,108,387	391,232	13.05
West South Central	7,496,965	834,209	8.95
Mountain	4,305,524	399,160	11.00
Pacific	11,877,984	984,171	8.30
Total	122,626,166	10,423,380	11.80

In this year the owners and operators of motor vehicles contributed $122,626,166, which was included as a part of the total highway income for the country. Averaging this amount over the 10,423,380 motor vehicles which were registered results in an average payment of $11.80 per motor vehicle. In the New England States the average was the highest, $16.40. This was wholly due to the higher scale of license fees charged because there was no gasoline tax in effect in that year in any of these States. In the Pacific States the highway burden placed upon the user of the road, $8.30, was the lowest. The variations are almost wholly due to the differences in the scale of license fees.

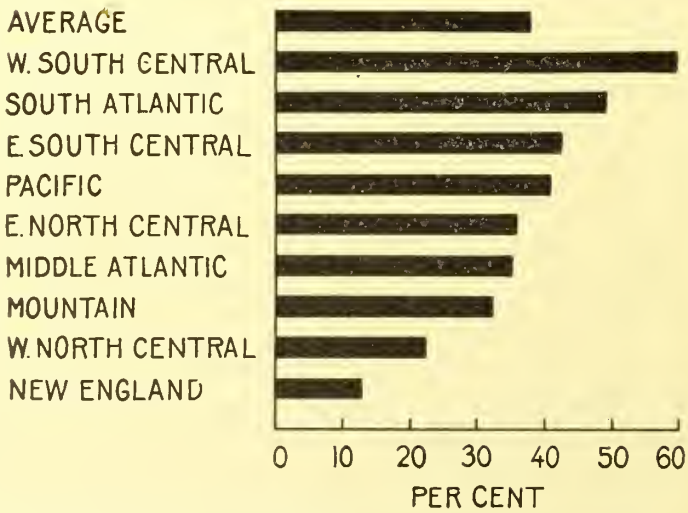


Ratio of motor-vehicle fees and gasoline taxes to total highway income, 1921, by geographic divisions

The ratio which the motor-vehicle revenues bore to the total highway incomes in the several sections of the country is shown below:

Division	Ratio of motor-vehicle fees and gasoline taxes to total highway income, 1921	Division	Ratio of motor-vehicle fees and gasoline taxes to total highway income, 1921
	Per cent		*Per cent*
New England	25.1	West South Central	5.4
Middle Atlantic	13.3	Mountain	7.0
East North Central	9.8	Pacific	11.6
West North Central	13.4		
South Atlantic	8.6	Average	10.6
East South Central	8.5		

For the country as a whole the motor vehicles contributed 10.6 per cent of the total highway income. The range was from 5.4 per cent in the West South Central States to 25.1 per cent in the New England States. Outside of the New England States the highest ratio of motor-vehicle revenues to total highway income was 13.4 per cent in the West North Central States.



Ratio of revenues derived from bond sales to total highway income, 1921, by geographic divisions

THIRTY-EIGHT PER CENT OF INCOME FROM BONDS

That part of the total highway income which was derived from the sale of bonds varies widely in the different States and geographical divisions. The ratio of the amounts of money thus raised for highway purposes to the total highway income is set forth in the following tabulation:

Division	Revenues derived from sale of bonds, 1921	
	Amount	Per cent of total highway income
New England	$5,889,745	12.7
Middle Atlantic	59,543,258	35.4
East North Central	101,550,318	36.1
West North Central	34,291,178	22.3
South Atlantic	67,406,730	49.0
East South Central	25,551,347	42.5
West South Central	82,127,751	59.3
Mountain	19,908,036	32.4
Pacific	41,840,910	41.0
Total	438,109,273	38.1

For the country as a whole $438,109,273, or 38.1 per cent, of the total highway income was obtained from bond issues. In the West South Central States the largest proportion of highway income, 59.3 per cent, was represented by State and local indebtedness. In the New England States the smallest part of the year's highway funds, 12.7 per cent, was raised in this manner. In all of the groups of States, with the exception of the New England States, the West North Central States, and the Mountain States, over a third of money devoted to highway work was obtained through bond issues. In only one of the divisions, the West South Central States, did the income from bonds amount to more than half of the year's total highway income.

The per capita obligations incurred through these bond issues for highway purposes vary in these several groups as shown in the following statement:

Division	Per capita highway bond issues, 1921	Division	Per capita highway bond issues, 1921
New England	$0.80	West South Central	$8.00
Middle Atlantic	2.68	Mountain	6.00
East North Central	4.70	Pacific	7.50
West North Central	2.75		
South Atlantic	4.80	Average	4.10
East South Central	2.90		

In the New England States because of the large population and the relatively small amount of bonds issued the per capita obligation of 80 cents was lower than in any other section. The highest per capita obligation was found in the West South Central States, where the income from bond issues amounted to $8 per capita.

FEDERAL AID AMOUNTS TO 7 PER CENT OF TOTAL HIGHWAY INCOME

To the total highway income of 1921 the Federal Government contributed $79,333,226, or 6.9 per cent. In the following tabulation are shown the proportions which these Federal funds bore to the total highway incomes of the several groups of States:

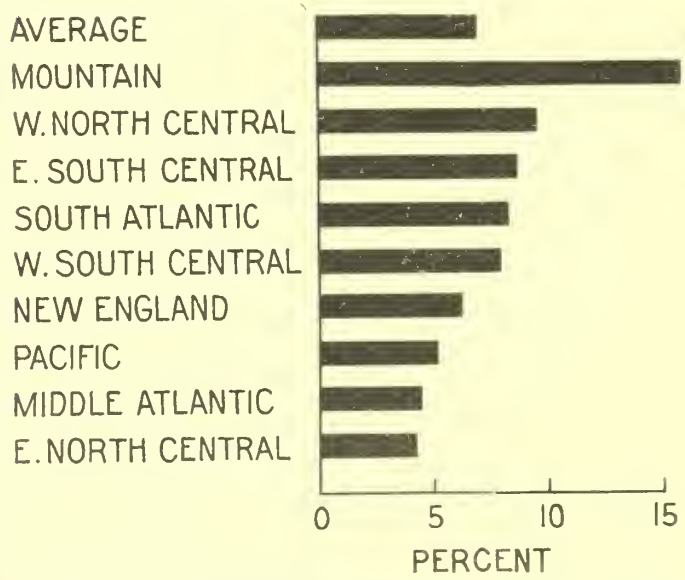
Division	Federal aid, 1921, per cent of total highway income	Division	Federal aid, 1921, per cent of total highway income
New England	6.2	West South Central	7.9
Middle Atlantic	4.4	Mountain	15.8
East North Central	4.2	Pacific	5.1
West North Central	9.5		
South Atlantic	8.2	Average	6.9
East South Central	8.7		

The variations in the ratios of Federal aid funds to the total highway income of these several groups of States depend upon the magnitude of the highway-construction programs and upon the amount of forest-road construction by the Federal Government in each of the States, and also upon the apportionments according to the provisions of the Federal law.

PROPERTY TAXES 44 PER CENT OF THE TOTAL INCOME

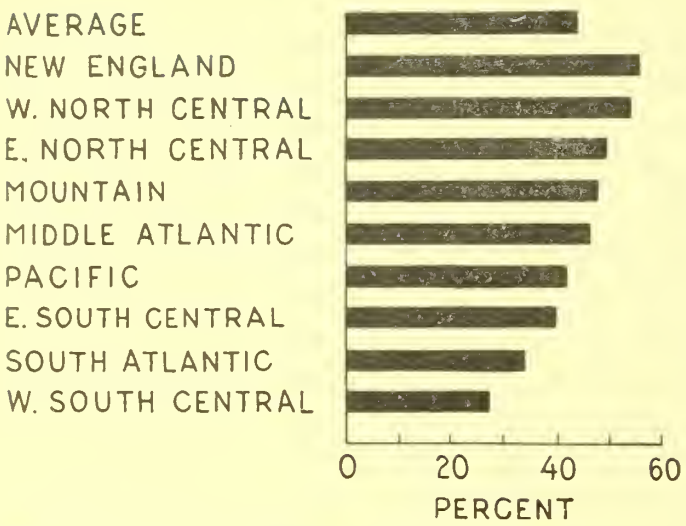
That part of the total highway income derived from general property and from other sources, chiefly taxes, amounted to $509,369,231, or 44.4 per cent of the total highway income. Its relation to the rest of the highway income in various groups of States is indicated in the following table:

Division	General property taxes and other sources of revenue for highway purposes, 1921	
	Amount	Per cent of total highway income
New England	$26,031,762	56.0
Middle Atlantic	78,980,242	46.9
East North Central	140,268,558	49.9
West North Central	84,140,831	54.8
South Atlantic	47,122,844	34.2
East South Central	24,339,475	40.3
West South Central	37,723,723	27.4
Mountain	27,422,857	44.8
Pacific	43,112,939	42.3
Total	509,369,231	44.4



Ratio of revenues derived from Federal aid and forest road funds to total highway income, 1921, by geographic divisions

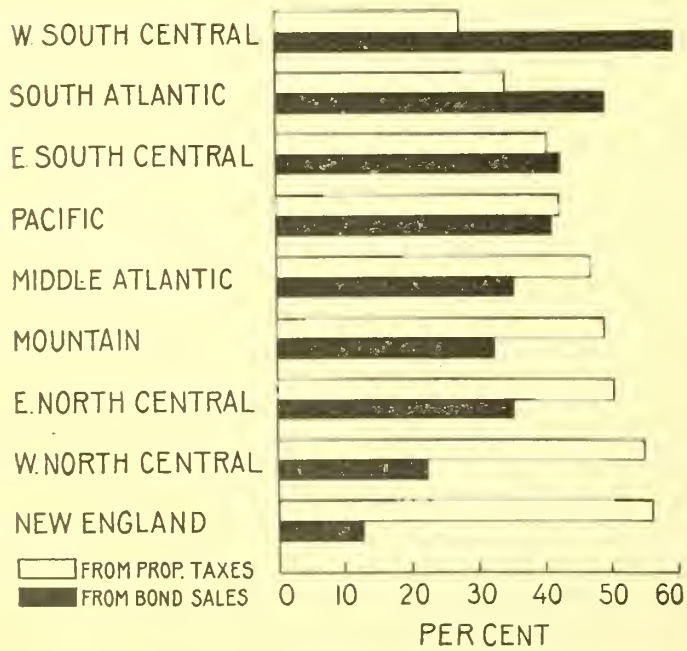
The largest amount of revenues raised from taxes is found in the East North Central States, where the direct contribution of taxpayers amounted to $140,268,558, just half of the total highway income. In the New England States the percentage of total highway income derived from taxes was the highest and in the West South Central States it was the lowest. The percentage variation in this case is the inverse of the percentage of highway income raised through bond issues as seen in the tabulation on page 6.



Ratio of revenues derived from property taxes to total highway income, 1921, by geographic divisions

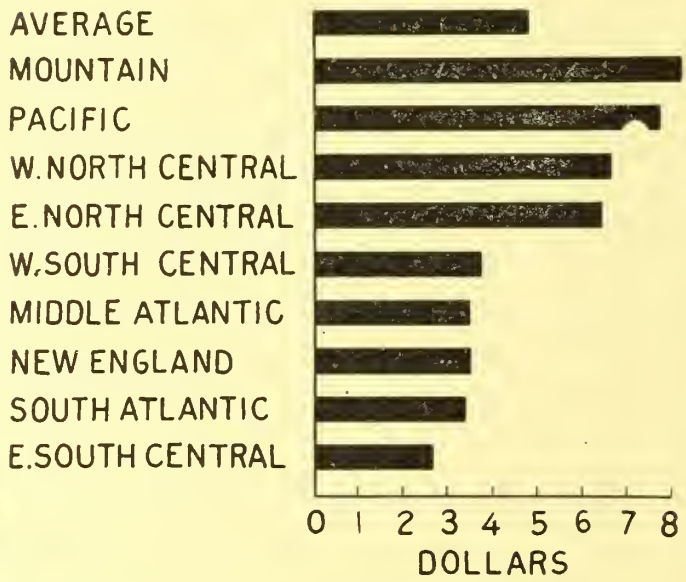
Division	Proportion of total highway income from general property taxes and bond issues	
	General property taxes	Bond issues
	Per cent	*Per cent*
New England	56.0	12.7
West North Central	54.8	22.3
East North Central	49.9	36.1
Mountain	44.8	32.4
Middle Atlantic	46.9	35.4
Pacific	42.3	41.0
East South Central	40.3	42.5
South Atlantic	34.2	49.0
West South Central	27.4	59.3

It will be observed that those groups of States which showed the highest percentages of the total highway income derived from taxes showed the lowest percentages of highway revenues derived from bond issues.



Comparison of highway income from general property taxes and bond sales, 1921, by geographic divisions

In the New England States 56 per cent of the total income was derived from taxes and only 12.7 per cent from the proceeds of bond issues. At the other extreme are found the West South Central States, which raised 59.3 per cent of their highway income through bond issues and only 27.4 per cent through general property taxes and other taxes. The fact that the West South Central States raised a larger percentage of their highway income through taxes than the New England States raised through bond issues, although the percentage of income raised through taxes of the New England States and the percentage raised by bond issues in the West South Central States are very nearly the same, is explained by the fact that the New England States raise a larger percentage of their highway income by taxation of motor vehicles. The New England States derived 25.1 per cent of their highway revenues from motor-vehicle fees, while the West South Central States derived but 5.4 per cent in this manner. It can be said in considering the sources of highway income in these different groups of States that the ratio of revenues derived from taxes varies inversely with the ratio derived from bond sales.



Per capita burden of highway income derived from general property taxes, 1921, by geographic divisions

PER CAPITA TAX BURDEN FOR HIGHWAYS LESS THAN FIVE DOLLARS

The burden which the collection of these taxes for highway purposes placed upon individuals is indicated in the following tabulation:

Division.	Per capita highway income derived from taxes	Division	Per capita highway income derived from taxes
New England	$3.52	West South Central	$3.70
Middle Atlantic	3.52	Mountain	8.20
East North Central	6.50	Pacific	7.75
West North Central	6.70		
South Atlantic	3.40	Average	4.82
East South Central	2.70		

The direct payments made by the public for rural highway construction and improvements aside from the revenues derived from other sources vary considerably

in the different sections of the country. In the East South Central States the direct payments of taxpayers amount to only $2.70 per capita; that is the lowest per capita payment. The highest per capita payment is found in the Mountain States, where the contribution from taxpayers is $8.20. It is of interest to note that the per capita burden of tax payments for highway purposes are exactly the same for the New England States and for the Middle Atlantic States. There is also a close coincidence in the per capita payments in both the East and the West North Central groups. For the country as a whole the per capita tax burden was $4.82.

In order to determine how much of a burden the taxes collected for highway purposes really are it is of interest to compare the per capita burden of the total taxes paid by the public and the per capita taxes which are collected for highway purposes. In the following tabulation this relationship is clearly set forth:

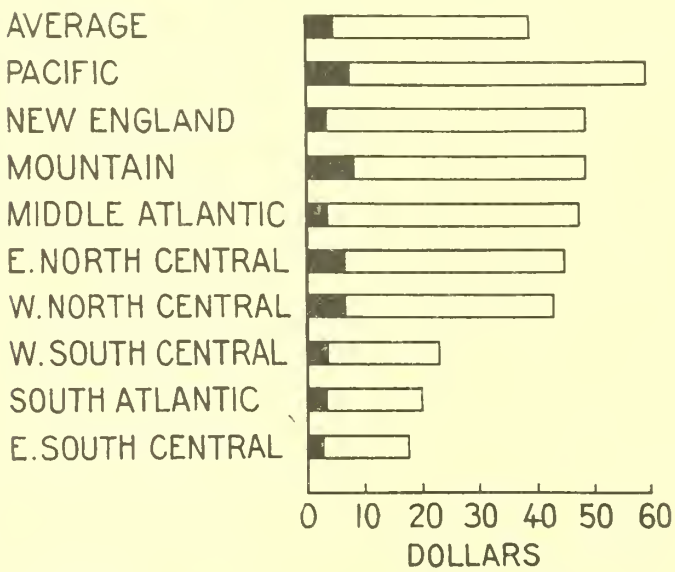
Division	Total taxes, fees, etc., per capita, 1921	Highway taxes, per capita	
		Amount	Per cent of total taxes
New England	$49.00	$3.52	7.2
Middle Atlantic	47.30	3.52	7.5
East North Central	45.20	6.50	14.4
West North Central	43.00	6.70	15.5
South Atlantic	20.10	3.40	16.8
East South Central	17.55	2.70	15.3
West South Central	23.00	3.70	16.1
Mountain	48.90	8.20	16.8
Pacific	59.20	7.75	13.1
Average	38.80	4.82	12.4

In comparing the relation which the per capita tax payments for highway purposes bears to the total tax payments, we note that the country can be divided roughly into two sections. In the New England and Middle Atlantic States the ratios are 7.2 and 7.5 per cent, respectively. In this northeastern section slightly over 7 per cent of the taxes collected accrue to the highway funds of the States. The per capita total taxes and the per capita highway taxes are quite uniform. In the rest of the country the ratio of per capita highway taxes to the per capita total taxes varies from 13.1 to 16.8 per cent, showing again that the relative burden of direct payments for highway purposes is quite uniform, although the per capita amounts differ considerably for the various sections.

The highway income of the country is, of course, called forth by the highway expenditures. The magnitude of these expenditures makes necessary the collection of funds sufficient to meet the demands made by highway construction and maintenance and to meet the payments of principal and interest on past bond issues. There are three variables which determine the burdens of these expenditures for individuals, namely, the amount of money spent per mile of road, the miles of roads constructed and maintained, and the population which has to make the financial contributions. In order to make comparisons which recognize each of these three variables it is important to determine what the expenditures have been per person per mile of road. If we take per capita expenditures alone, we can, it is true, make comparisons as to what the outlays for highway purposes have been in different sections and what the individual burden has been. This does not, however, tell the whole story. A relatively high per capita expenditure may mean any of a number of conditions or a combination of such conditions; it may mean that the per capita expenditure is relatively high because of the small population in a State or in a group of States; it may be relatively high because of a very large road mileage upon which improvements have been made; it may be high because high type roads have been constructed although the road mileage may be relatively small in amount; or it may be due to a combination of these three factors.

AVERAGE EXPENDITURE PER PERSON 1 CENT FOR 200 MILES OF ROAD

In 1921 the total highway expenditures for all rural highway purposes were $1,036,587,772. In the following tabulation the resulting per capita expenditure in each of the sections is shown in relation to the total mileage of road in the corresponding sections with the purpose of bringing out the per capita expense per mile of road:



Comparison of per capita highway taxes with total per capita taxes, State and local, by geographic divisions

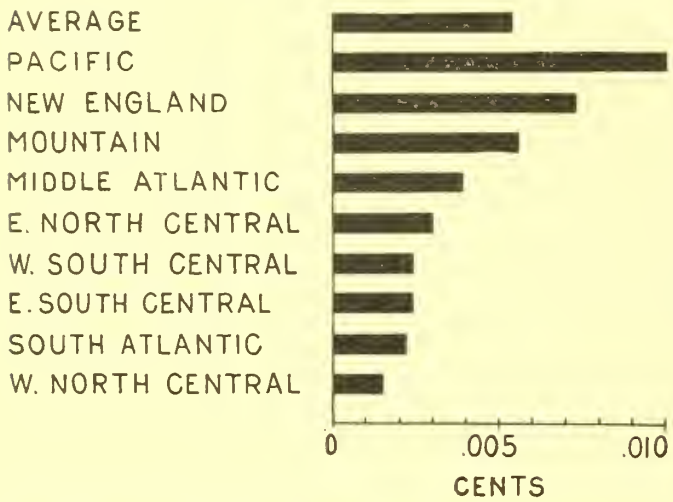
Division	Mileage of rural highway	Highway expenditures per capita	Highway expenditures per person per mile of road
			Cents
New England	83,296	$6.10	0.0073
Middle Atlantic	186,935	7.20	.0039
East North Central	412,753	12.40	.0030
West North Central	759,820	11.80	.0015
South Atlantic	365,567	7.70	.0022
East South Central	242,745	5.75	.0024
West South Central	416,617	10.20	.002a
Mountain	306,382	17.15	.0056
Pacific	167,180	16.90	.0101
Total	2,941,294	[1] 9.80	[1] .0054

[1] Average.

The highway expenditures per person per mile, while they averaged 0.0054 cent, ranged from 0.0015 cent in the West North Central States to 0.0101 cent per person per mile in the Pacific States. A more direct comparison can be made by arranging the several groups in order from the lowest to the highest.

Division	Expenditures per person per mile	Percentage relationship
West North Central	0.0015	100
South Atlantic	.0022	147
East South Central	.0024	160
West South Central	.0024	160
East North Central	.0030	200
Middle Atlantic	.0039	260
Mountain	.0056	370
New England	.0073	480
Pacific	.0101	670

An examination of these expenditures per person per mile reveals the relative significance of expenditures for highway improvements. This is seen, for example, in comparing the West North Central group with the East North Central group. The per capita highway expenditures of the West North Central division were $11.80 and for the East North Central



Highway expenditures per person per mile of road, 1921, by geographic divisions

group $12.40. But when the comparison is based upon the expenditures per person per mile of road a wholly different result is obtained. On this basis the highway expenditures per person per mile in the East North Central States were 100 per cent above those of the West North Central States. The highest expenditure per person per mile of road was found in the Pacific States, where it amounted to 0.0101 cent, or 670 per cent of that in the West North Central States.

STATE HIGHWAY DEPARTMENTS RESPONSIBLE FOR ONLY 40 PER CENT OF STATE EXPENDITURE

The rural highway income which has been analyzed thus far is the income of the whole country used for construction and maintenance of all rural roads, State, county, and local. Only a part of this income is spent by State highway departments and by the direction or under the supervision of such departments. The total highway expenditures for 1921 were $1,036,587,772; of this amount only $413,241,662, or 39.8 per cent, was spent by or under the supervision of State highway departments. The remaining 60.2 per cent of the expenditure was made by counties, townships, and highway districts. The ratios of expenditures made by State highway departments to the total highway expenditures in the several groups of States are set forth in the following table:

Division	Total highway expenditure (1921)	Highway expenditures by or under State highway departments	
		Amount	Per cent of total
New England	$45,361,397	$26,757,690	59.0
Middle Atlantic	160,470,634	101,477,088	63.5
East North Central	267,988,153	68,512,054	25.6
West North Central	148,865,686	47,143,661	31.7
South Atlantic	106,656,946	51,181,157	48.0
East South Central	51,236,234	16,620,731	32.4
West South Central	103,820,179	33,951,911	32.7
Mountain	58,157,949	28,001,863	48.2
Pacific	94,030,614	39,595,509	42.0
Total	1,036,587,772	413,241,664	39.8

It is obvious from this table that outside of the New England and Middle Atlantic States the major part of the highway expenditure is made by counties and local units. In the East North Central States, in which section or group of States the highway expenditures were greater than in any other group, the expenditures made by or under the supervision of the State highway departments were approximately only 25 per cent of the total highway expenditures. In the Middle Atlantic States the money expended by State departments was 63.5 per cent of the total, the largest in any group. By applying these percentages to the per capita taxes paid for highway improvements in the different groups of States we are able to ascertain the per capita burden which these State expenditures entail.

Group	Total highway income derived from taxes per capita	Highway expenditures made by State highway departments, per cent of total	Highway income derived from taxes and used by State highway departments, per capita
New England	$3.52	59.0	$2.08
Middle Atlantic	3.52	63.5	2.24
East North Central	6.50	25.6	1.66
West North Central	6.70	31.7	2.12
South Atlantic	3.40	48.0	1.63
East South Central	2.70	32.4	.88
West South Central	3.70	32.7	1.21
Mountain	8.20	48.2	3.95
Pacific	7.75	42.0	3.25
Average	4.82	39.8	1.94

The per capita burden of the total highway income derived from taxes was $4.82 for the country as a whole. It is of significance to note that only $1.94 of this per capita tax was used by State highway departments. In the East South Central States less than a dollar per capita of the taxes collected for highway purposes was turned over to State departments for expenditure.

HIGHWAY INCOME QUADRUPLED SINCE 1914

It is impossible to obtain complete data covering either the amount of money expended for highway purposes or the income devoted to such purposes year by year. Surveys, however, were made by the Bureau of Public Roads for the years 1904 and 1914, and it is possible to compare the figures for these years with those for 1921. It is a matter of common knowledge that the development of highway work was very great during the period between 1904 and 1921. In 1904 the

total highway income of States, counties, and local districts was $79,623,594; in 1914 it had increased to $240,263,766, and in 1921 it had risen to $1,119,437,896. It should be borne in mind that these amounts include the total highway funds irrespective of their source. They include tax funds, motor-vehicle revenues, proceeds of bond issues, and Federal-aid contributions. In the following tabulation the highway income is shown by groups of States for the years 1904, 1914, and 1921. For purposes of comparison the amounts for the various sections in 1904 and 1921 are shown as percentages of the 1914 amounts.

Division	Total highway income					
	1904		1914		1921	
	Amount	Per cent	Amount	Per cent	Amount	Per cent
New England	$7,385,753	48	$15,435,744	100	$46,455,284	301
Middle Atlantic	13,854,590	34	40,864,831	100	168,305,433	412
East North Central	19,613,190	35	56,445,180	100	281,139,024	500
West North Central	10,482,195	32	33,120,011	100	153,642,716	462
South Atlantic	7,308,145	30	24,428,952	100	137,657,698	565
East South Central	7,022,385	55	12,754,577	100	60,280,684	572
West South Central	7,260,146	47	15,333,027	100	138,504,160	905
Mountain	2,309,349	25	9,454,277	100	61,367,959	650
Pacific	4,389,841	13	32,427,167	100	102,084,938	315
Total	79,623,594	33	240,263,766	100	1,119,437,896	465

In 1914 the income for highway purposes was practically three times the amount devoted to those purposes in 1904, and in 1921 it was over four times as large as in 1914. The demand for highway construction and improvement in 1904 had not yet been stimulated by the use of the automobile. By 1914 the motor vehicle had become an important factor, and by 1921 it had become the all-important factor both on account of the extension of the use of the motor vehicle and the large increase in the number in service. It is to be noted that the greatest increases in highway funds between 1904 and 1914 were in the Mountain and Pacific States. In the other groups of States the increases were more or less uniform.

While the highway income for 1921 averaged 465 per cent of the 1914 income, the Western States again exceeded all the others in their increases. In the Mountain States the highway income for 1921 was 650 per cent of the amount reported for that purpose in 1914, and in the West South Central division it was 905 per cent. In the other States the 1921 income figures ranged from 301 per cent of the 1914 figures in the New England States to 565 per cent in the South Atlantic States.

INCREASE IN ROAD INCOME PARTLY DUE TO REDUCED PURCHASING POWER OF THE DOLLAR

In making these comparisons of highway income we must not lose sight of the fact that between 1914 and 1921 there was also a marked advance in the general price level. The large increase of money expended and income received for highway purposes from 1914 to 1921 was caused not only by the more extensive program of construction and maintenance but by the decrease in the purchasing power of the dollar. The Bureau of Labor index numbers indicate that over that period there was an increase of 50 per cent in wholesale prices. It is believed that highway construction costs taking into account labor and materials were subject to at least as great an increase. In other words, if there had been no change in the price level the income necessary to meet the highway expenditures of 1921 would have been about $746,000,000 or an increase of approximately 320 per cent over that of 1914.

In 1914 there were 1,711,339 motor-vehicle registrations as compared with 10,463,295 in 1921, an increase of 610 per cent. This large increase in the use of the automobile no doubt provided the incentive for the increased highway expenditures which made necessary the larger highway revenues.

In the following table is shown a comparison of the total per capita highway incomes for these years for which the data are available:

Division	Per Capita highway income					
	1904		1914		1921	
	Amount[1]	Per cent	Amount[2]	Per cent	Amount[3]	Per cent.
New England	$1.32	56	$2.35	100	$6.25	266
Middle Atlantic	.90	42	2.12	100	7.50	354
East North Central	1.23	40	3.09	100	13.10	424
West North Central	1.01	32	3.14	100	12.25	390
South Atlantic	.72	35	2.06	100	9.85	480
East South Central	.93	61	1.52	100	6.80	448
West South Central	1.11	63	1.75	100	13.50	770
Mountain	1.38	38	3.60	100	18.40	510
Pacific	1.82	23	7.75	100	18.30	236
All States	1.05	40	2.64	100	10.90	418

[1] Based on 1900 population. [2] Based on 1910 population. [3] Based on 1920 population

It will be noted that the percentages in this table differ somewhat from those with respect to total highway incomes. This is largely brought about by changes in the population figures. In 1904 the per capita highway income was $1.05; in 1914 it had risen to $2.61; and in 1921 it was $10.90, which was 418 per cent of the 1914 figure. The per capita income was the largest in the Mountain and in the Pacific divisions. The largest increase over 1914 was in the West South Central States, where the per capita income in 1914 was only $1.75, and in 1921 $13.50, an increase of 770 per cent. The greatest increase in per capita income between 1904 and 1921 was in the South Atlantic States where in 1904, it was only 72 cents and in 1921 $9.85, an increase of 1,415 per cent.

THE USE OF BONDS BY COUNTIES AND STATES

Up to this point the analysis has been confined to a determination of the sources of income in the various groups of States, including the income of State highway departments and counties and local districts. The data compiled by the Bureau of Public Roads show that for 1921 the counties, townships, and districts had an income of $743,493,212 for highway purposes as compared with an income of $405,944,684 which were the funds under the control of State highway departments. The total revenues raised through the sale of bonds has already been set forth. In this connection, however, it is important to ascertain the extent to which bond issues were used to raise funds by the States and by the counties and local subdivisions.

The data show that counties, townships, and districts derived $322,613,529, or 43.5 per cent, of their total highway income from the sale of bonds in 1921. This relation is shown for the various groups in the tabulation following.

Division	Total highway income of counties, townships, and districts	Income derived from bond sales	
		Amount	Per cent of total
New England	$19,441,981	$3,504,650	18.0
Middle Atlantic	58,768,442	14,414,467	24.5
East North Central	221,080,834	91,392,840	41.3
West North Central	107,313,301	28,104,534	26.4
South Atlantic	76,590,092	39,189,230	51.2
East South Central	46,329,099	24,464,597	53.0
West South Central	117,492,554	81,443,272	69.0
Mountain	30,675,600	14,338,388	46.7
Pacific	65,801,309	25,761,551	38.6
Total	743,493,212	322,613,529	43.5

For purposes of comparison the following table shows the percentage of the State highway income derived from bond sales:

Division	Total funds under control State highway departments	State highway income derived from bond sales	
		Amount	Per cent of total
New England	$27,013,303	$2,385,095	8.8
Middle Atlantic	109,536,991	45,128,791	41.3
East North Central	60,058,190	10,157,478	16.9
West North Central	46,329,415	7,708,079	17.0
South Atlantic	61,067,606	27,112,708	44.5
East South Central	13,951,585		
West South Central	21,011,606	684,479	3.3
Mountain	30,692,359	5,569,648	18.2
Pacific	36,283,629	16,079,359	44.1
Total	405,944,684	114,825,637	28.1

In several of the sections, it will be noted, a very substantial part of the county and local income was derived from bond issues. In the West South Central States 69 per cent of the local highway funds were procured in this manner. In this section and in the South Atlantic States and in the East South Central States the proceeds derived from bond sales amounted to over 50 per cent of the local highway revenues. In the New England States, on the other hand, the bond sales amounted to only 18.05 per cent of the total local highway income, and this was the smallest percentage for the country.

It will be seen that the proportion of the income available to the State highway departments which was derived from bond sales was only 28.1 per cent as compared with 43.5 per cent in the case of the counties and local units. The South Atlantic, the Pacific, and the Middle Atlantic States derived from 41 to 44.5 per cent of the State highway revenues from bonds; in the other groups the percentage was considerably less; and it is worthy of especial note that none of the East South Central States raised any of their State funds by bond issues. In a number of the States the failure to employ this source of income is due to prohibitory clauses in their constitutions.

What the future trend will be as to the relativity between the different sources of highway income is hard to foretell. It has already been noted that there has been a large increase in both the absolute and relative amount of funds derived from motor-vehicle license fees and gasoline taxes. The present indications are that the revenues derived from these sources will increase, especially because so many States have recently enacted gasoline tax laws and because some of the States which have been collecting gasoline taxes contemplate increasing the rates charged. If we assume that there will be no increase in highway expenditures, the additional funds derived from motor-vehicle revenues may result in a decrease in the amount collected as taxes or a decrease in the amount raised by bond issues or in both.

PENNSYLVANIA HIGHWAY TRANSPORT SURVEY

INVESTIGATION SIMILAR TO CONNECTICUT STUDY UNDER WAY

STATE HIGHWAY DEPARTMENT AND FEDERAL BUREAU COOPERATING

A HIGHWAY transportation survey similar in its purposes and general methods to the survey conducted in Connecticut and described in the March issue of PUBLIC ROADS is well under way in Pennsylvania. The survey, which is being conducted by the United States Bureau of Public Roads in cooperation with the Pennsylvania Department of Highways, was begun in November, 1923, and will be continued through October, 1924.

The adjustment of the State highway system to the economic needs of the traffic is a practical problem of the first importance in every State. It is a fundamental principle of economic highway design that no road should be improved to a degree in excess of its earning capacity. The return to the public in the form of economic transportation is the sole measure of the worth of the improvement. Where an effort is made to conform to this principle, the first question asked by the designing engineer is: What will be the volume of traffic when the road is improved? A mere census of the traffic before the road is improved will not answer this question. Is it possible by consideration of the density of the population on the territory adjacent to the road, combined with a study of the volume and character of the production of the territory, its wealth and the number of motor vehicles owned in the tributary area, to develop a basis for such a forecast? This is one of the subjects which will receive the attention of the cooperating agencies in the Pennsylvania survey.

Other practical facts sought in the survey are:

The maximum loading and maximum size of vehicles and the frequency of critical loads as an index of pavement width and design requirements.

The effect of congestion at intersections and "bottle necks" upon the rate of traffic flow.

The extent to which the improvement of old roads or the opening of new traffic routes is economically justified.

The type and volume of traffic using the various highways of the State as an index to the allocation of highway construction and maintenance funds.

The relation between traffic loads and density and the costs of highway construction and maintenance.

The amount and frequency of motor truck overloading, the distribution of load to the front and rear axles of the vehicles, the depth and width of tires.

The cost of various types of improvements, such as relocations, grade reductions, elimination of grade crossings and elimination of traffic "bottle necks" with the estimated saving in transportation costs resulting from such improvements.

The earning value of the State highway system in comparison with the present worth of the system using the replacement value minus depreciation as the basis of computing present worth.

The classification of Pennsylvania highways as industrial, high, medium or low-type traffic routes based on (1) total passenger car and motor truck density and (2) motor truck capacities. Design is influenced by present and anticipated capacity usage. Routes having a large proportion of trucks of small capacity using the highway will not require the same construction as where heavy capacities prevail.

BASIC ECONOMIC DATA SOUGHT

Besides these facts, each of which has immediate value in connection with the design and administration of the State highways, it is hoped to develop a large amount of basic economic data, such as the tonnage shipped by motor truck, marketing methods, and the relation of highway transportation to other methods of transportation. Other information of this character sought by the survey includes the following:

1. The mileage zones of motor-truck haulage and the relation of the type of commodity hauled to such zones.

2. The net tonnage of freight transported by regular and irregular trucking operators.

3. The situs of ownership of passenger cars and motor trucks operating over the State highway system.

4. The value of the net tonnage hauled over the system by motor truck.

5. The origin and destination of the commodities transported over the highway system and the character of origin and destination; i. e., whether the movement is between farm and railroad, from farm to city merchant, etc.

6. The relation of motor-truck transportation to other methods of transportation, particularly as to competition, rates, operating schedules, and delivery time in the short, middle-distance, and long-haul zones.

7. The data concerning haulage practices of truck operators and the tonnage transported by motor truck between various cities and areas which are of value to governmental agencies charged with the regulation and control of highway transportation.

8. The usage of passenger cars for business and nonbusiness purposes.

In order to secure the desired information, four divisions have been formed with headquarters in Philadelphia, Wilkes-Barre, Franklin, and Pittsburgh. The operating personnel in each of these divisions is headed by a division supervisor, who reports directly to the field manager or his assistant at Harrisburg. All costs of the survey are shared equally by the Pennsylvania department and the Federal bureau.

TRAFFIC OBSERVED AT 312 STATIONS

Three hundred and twelve observation stations have been established, 85 per cent of them on primary roads and 15 per cent on secondary roads. At 78 of these stations the information secured includes complete weight data; the others, known as recording stations, have no weighing facilities. Pit scales have been installed at 18 of the most important weighing stations. The other 60 are equipped with portable Berry scales.

In each division the personnel is divided into parties, which, according to the stations to which they are regularly assigned, are known as pit scale parties, Berry scale parties, recording, and night parties. Thus, in the Philadelphia division, there are 2 pit scale parties, 1 Berry scale party, 3 recording parties, and 1 night party. In the Wilkes-Barre and Franklin divisions there are 2 recording parties, 1 Berry scale party and 1 night party; and in the Pittsburgh division there are 2 Berry scale parties, 2 recording parties, 1 pit scale party, and 1 night party.

Each recording party has 26 stations at which it operates each month, working every day at a different station. A pit scale party remains at each station a week and completes its cycle in 6 weeks, there being 18 stations and 3 parties. Each Berry scale party operates 12 stations, and by remaining at each station 2 days completes its cycle in about a month. In addition to these parties, which are occupied mainly in collecting the motor-truck data, a single passenger car party moves from station to station and division to division to obtain the prescribed information with reference to passenger-car traffic, and the night party in each division, operating at key stations on selected routes, secures the information with regard to the night traffic which will be used to extend the day observations to a 24-hour basis.

The arrangement of the stations is such that the weight data secured at the 78 weighing stations can be applied to the traffic observed at the 234 recording stations, and the schedules of the various parties are so devised that there is no duplication of the records.

HIGHWAY FREIGHT TRAINS A FEATURE OF NIGHT TRAFFIC BETWEEN PHILADELPHIA AND NEW YORK

The observations of the night parties are especially important in the Philadelphia division because of the heavy night movement of trucks between Philadelphia and New York. Sixty per cent of the movement between these cities takes place at night, much of it in convoys under the charge of a truck master. As many as nine trucks have been counted in these highway freight trains.

Several items of information are being obtained in this survey which have not been secured in any previous survey. For example, the information in regard to origin and destination of truck shipments embraces not only the terminal points but the character of these points, so that it will be possible to describe the exact character of each movement as a farm-to-railroad shipment, residence-to-residence, manufacturer-to-jobber, etc. The observations with respect to the situs of motor-vehicle ownership also constitute a new class of information; and the passenger bus data, including origin, destination, capacity, number of passengers and frequency of trips will be more complete than any similar information previously secured.

VOLUME CHANGE A MEASURE OF ALKALI ACTION

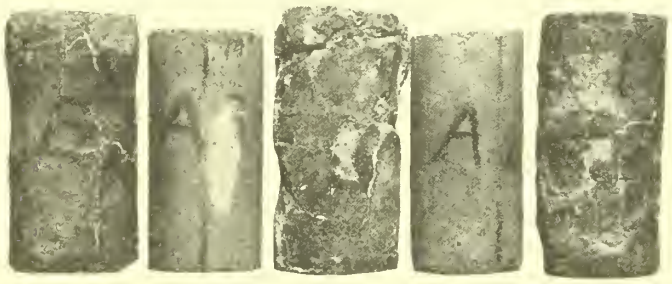
EFFECT ON CONCRETE MEASURED BY CHANGES IN LENGTH

By DALTON G. MILLER, Senior Drainage Engineer, U. S. Bureau of Public Roads

EDITOR'S NOTE. The May issue of PUBLIC ROADS contained reports of investigations by the Bureau of Public Roads to determine the effect of alkaline mixing water on the strength of concrete, and to develop protective treatments with tars and paraffin. While the experiments upon which the following report is based have for their principal purpose the improvement of the quality of farm drain tile, the results of the studies may be applied also to concrete pipe culverts. The particular relation described has no direct bearing upon design, but should have useful application in the laboratory.

The experiments described were conducted at the drain tile laboratory, University Farm, St. Paul, Minn., by the Department of Agriculture of the University of Minnesota, the Department of Drainage and Waters of Minnesota, and the division of agricultural engineering of the United States Bureau of Public Roads.

THE SO-CALLED alkali problem in connection with the use of concrete drain tile in Minnesota was early resolved into a question of the effect of sulphates of magnesium and sodium, and as the action on concrete of each of these salts is accompanied by a very decided increase in volume it was decided to use in the laboratory both the actual increase and the rate of increase as indicated by length changes, as a basis for comparing the behavior of different specimens in the same solution as well as the effect of different solutions on similar specimens.



The increase in volume caused by immersion in magnesium sulphate solution is here plainly illustrated. The three swollen cylinders have been immersed in a 1 per cent solution for 38 weeks. The other two from the same lot have been kept in distilled water for the same length of time. All cylinders are 1:3 mix, normal consistency, standard Ottawa sand

Heretofore it has been the practice in work of this general nature to depend upon the appearance of the specimens, supplemented by compression or tension tests to determine the effect of the chemical action. It is obvious that the mere appearance of a specimen is not entirely satisfactory as a criterion, and while compression and tension tests are of course fundamental, and consequently must finally form the basis for any conclusions, yet such tests are subject to certain limitations in studies that have to do with the deterioration of concrete, since the life of a specimen is of course ended when once it is broken. In ordinary laboratory work on concrete the testing of a few specimens more or less is not of great moment; but where the specimens are kept in solutions for long periods and where it is necessary to renew the solutions at short intervals, it becomes essential to reduce so far as possible without impairing the value of the work—the number of specimens tested.

In the Minnesota investigations all experimental pieces are kept in solutions in earthenware jars, and

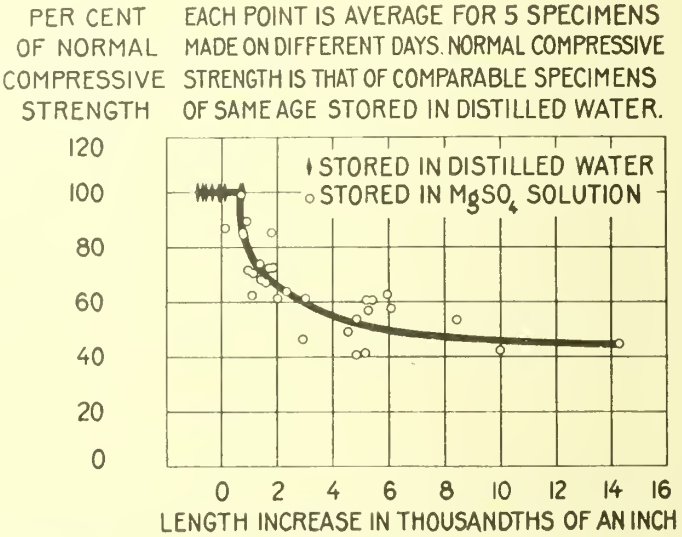

PER CENT OF NORMAL COMPRESSIVE STRENGTH
EACH POINT IS AVERAGE FOR 5 SPECIMENS MADE ON DIFFERENT DAYS. NORMAL COMPRESSIVE STRENGTH IS THAT OF COMPARABLE SPECIMENS OF SAME AGE STORED IN DISTILLED WATER.
STORED IN DISTILLED WATER
STORED IN MgSO4 SOLUTION
120
100
80
60
40
20
0
0 2 4 6 8 10 12 14 16
LENGTH INCREASE IN THOUSANDTHS OF AN INCH

FIG. 1.—Relation between length changes and compressive strength of 2 by 4 inch Portland cement mortar and concrete cylinders immersed in solutions of magnesium sulphate

the solutions are changed weekly. In order to insure consistent results it has been found that this change must be made at frequent intervals. In fact the ideal arrangement would be to use a constantly changing solution, as this would most nearly represent field conditions. Two by four inch cylinders have been used in the work, as this size reduced to a minimum the volume of concrete in all jars. Moreover, the 2 inch diameter roughly approximates the thickness of the wall of the average size of tile used in the larger drains in Minnesota.

The cylinders are made in batches of nine, each series consisting of five batches made on different days. In each end of three cylinders of each batch a 1-inch brass screw is set in neat cement. Changes in length are determined by measurements between screw heads. The laboratory standard concrete specimen is mixed 1:3 by volume, normal consistency, with the aggregate graded to produce a fineness modulus of 4.67, the largest pebbles used passing a three-eighths inch screen.

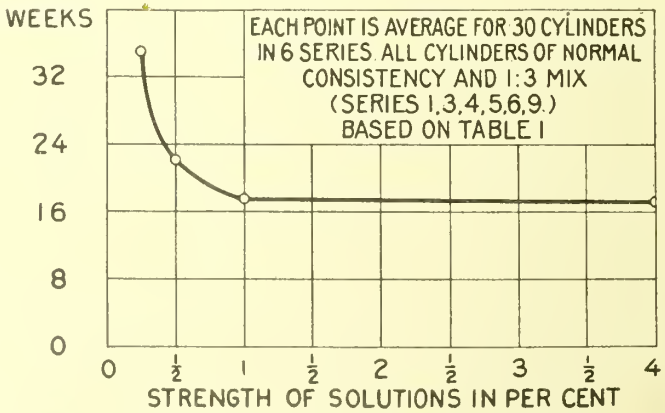

WEEKS
EACH POINT IS AVERAGE FOR 30 CYLINDERS IN 6 SERIES. ALL CYLINDERS OF NORMAL CONSISTENCY AND 1:3 MIX (SERIES 1,3,4,5,6,9.) BASED ON TABLE I
32
24
16
8
0
0 ½ 1 ½ 2 ½ 3 ½ 4
STRENGTH OF SOLUTIONS IN PER CENT

FIG. 2.—Time required for 2 by 4 inch Portland cement mortar and concrete cylinders in magnesium sulphate solutions of different strength to increase 0.010 inch

The cylinders are cured one day in the moist closet and 20 days in distilled water, after which they are placed in the solutions. Ten or fifteen cylinders have ordinarily been stored in each jar, the present practice being to start a series with 15 cylinders to a jar. In all cases the quantity of solution in a jar is 10 liters (2.6 gallons). About 10,000 cylinders have been made to date and this number is being constantly increased.

INCREASE IN LENGTH INDICATES LOSS IN STRENGTH

The results of some of the earlier work in the laboratory are shown in Tables 1 and 2 and by the curves in Figures 1, 2, and 3. Figure 1 is based on tests of some 200 cylinders. The points from which this curve was developed represent length changes of cylinders with screws and compression tests of blank-end cylinders from the same batches; consequently the relation between length change and strength ratio is not exact. Also the ends of many of the cylinders in the solutions become quite soft and crumble badly at the edges, rendering compression results more erratic than usual even after capping with plaster of Paris. It is evident, however, that any appreciable increase in length is indicative of loss of strength, as those cylinders stored in distilled water generally show no tendency to expand, but, on the contrary, in many cases slightly contract. Study of Figure 1 indicates that an increase in length of 0.10 inch in a 4-inch specimen means a loss in strength of 50 per cent or slightly more.

One of the interesting phases of the work so far has been that of the relative effects of solutions of different strengths on the cylinders. It has been found that the destructive effect of a 4 per cent solution is not much

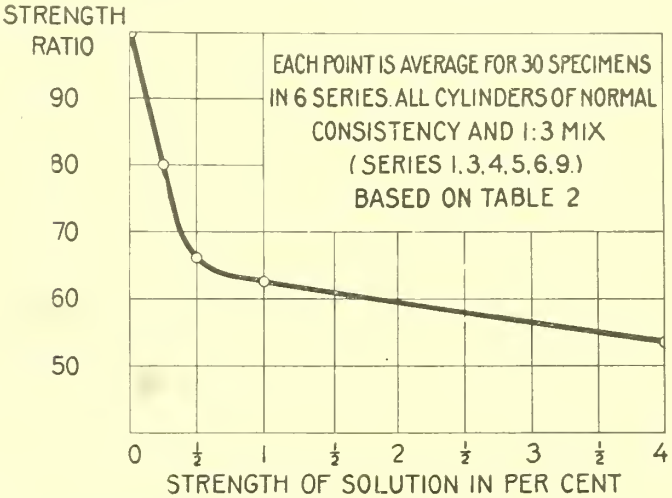


FIG. 3.—Strength ratio of 2 by 4 inch Portland cement mortar and concrete cylinders after immersion in magnesium sulphate solutions for periods averaging 15 weeks

greater than that of a 1 per cent solution, although for solutions below 1 per cent the action is approximately proportional to the strength of the solution. Because

(Continued on page 17)

TABLE 1.—*Time required for 2 by 4 inch Portland cement mortar and concrete cylinders immersed in magnesium sulphate solutions to increase in length 0.010 inch* [1]

Series	Fineness moduli	Compressive strength		Absorption at 21 days	Strength of solutions				Remarks
		7 days	28 days		One-fourth per cent	One-half per cent	1 per cent	4 per cent	
					Time required to increase 0.010 inch				
		Lbs. per sq. in.	*Lbs. per sq. in.*	*Per cent*	*Weeks*	*Weeks*	*Weeks*	*Weeks*	
1	3.00	1,850	2,840	7.1	25.6	15.0	12.9	15.8	Standard Ottawa sand.
3	3.00	1,370	2,180	11.8	32.1	14.5	10.8	10.4	Minnesota sand, screened 20–30.
4	3.00	1,640	2,470	9.3	21.9	12.5	10.8	10.6	Minnesota sand, screened 20–30, shale removed.
5	3.10	1,310	3,150	8.6	34.2	22.7	15.6	15.1	Minnesota sand, pit run.
6	4.67	2,700	4,250	6.2	44.5	35.4	23.8	19.8	Minnesota sand and pebbles passing three-eighths inch screen.
9	4.67	3,340	4,900	5.3	50.0	32.1	30.5	30.5	Minnesota sand, same as series 6, except shale removed.
Average	3.57	2,035	3,300	8.1	34.8	22.0	17.4	17.0	

[1] Each result is average for five cylinders of normal consistency and 1:3 mix made and measured on different days.

TABLE 2.—*Effect of different strength of solutions of magnesium sulphate on compressive strength of 2 by 4 inch Portland cement mortar and concrete cylinders of normal consistency and 1:3 mix* [1]

Series	Age	Strength of solutions									
		0 per cent	One-fourth per cent	One-half per cent	1 per cent	4 per cent	0 per cent	One-fourth per cent	One-half per cent	1 per cent	4 per cent
		Breaking strength					Strength ratios				
	Weeks	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*					
1	17	3,871	2,820	2,425	1,730	1,900	100.0	72.9	62.6	44.7	49.0
3	12	2,450	2,120	1,555	1,490	1,010	100.0	86.6	63.4	60.9	41.2
4	12	2,680	1,830	1,240	1,530	1,090	100.0	68.3	46.3	57.1	40.7
5	16	4,140	2,910	2,550	2,515	2,235	100.0	70.3	61.5	60.7	53.9
6	16	4,095	4,060	3,660	3,481	2,970	100.0	99.1	89.4	85.0	72.6
9	17	5,995	5,095	4,435	4,020	3,720	100.0	85.0	74.0	67.1	62.1
Average	15							80.4	66.2	62.6	53.3

[1] Each result is average for five cylinders made on different days.

OBSERVATIONS ON THE OPERATION OF MATERIAL TESTING LABORATORIES

By F. H. JACKSON, Senior Assistant Testing Engineer and C. E. PROUDLEY, Junior Assistant Testing Engineer, United States Bureau of Public Roads

DURING the last two construction seasons the writers, representing the United States Bureau of Public Roads, have visited a number of laboratories engaged in the testing of materials used in the construction of Federal-aid roads. The visits were made with the purpose of ascertaining to what extent these laboratories were following approved methods of testing as recommended by the Committee on Tests and Investigations of the American Association of State Highway Officials and approved by the Bureau of Public Roads. Fifty-five laboratories located in 45 States were inspected during the two seasons.

Since the passage of the Federal aid road act in 1916 the Bureau of Public Roads has required that all materials used in Federal-aid roads be tested for conformity with specifications. Prior to 1916 very few of the States were provided with proper facilities for carrying on this work, and it was necessary for the bureau at first to do a considerable amount of testing in its own laboratory. Within the last three or four years, however, it has been possible for a number of the States to install their own testing laboratories. Others have found it possible to enter into cooperative agreements with universities or technical schools, and still others have been able to obtain the services of existing commercial testing laboratories. During the 1923 season the bureau records show that there were 28 State and 24 university and 35 commercial laboratories engaged wholly or in part in the routine testing of Federal-aid road materials.

In the organization of the newer laboratories it has not always been possible to secure experienced men as operators, nor has it always been possible to install adequate equipment. These unsatisfactory conditions have resulted in some cases in the reporting of erroneous test results, which are naturally accepted at their face value by engineers in charge of construction. Thus they may lead to the acceptance of defective material or the rejection of acceptable material. It is obvious that the value of testing in general hinges on the accuracy with which the tests are made, and for this reason the Bureau of Public Roads is directly interested in assisting road materials laboratories in following standard methods. It may be of interest to highway and testing engineers to call attention briefly to some of the points observed in connection with the inspections made thus far.

THE LABORATORY FACILITIES

All enterprises are subject to a period of growth at the beginning of which they will be found frequently in a rather unsatisfactory state. Road materials testing laboratories are not exceptional in this respect, and the bureau's survey showed that in the character and amount of space and equipment available not less than 20 per cent of them were totally unsatisfactory for their purpose and only about 30 per cent were in what might be termed "first-class condition." We are considering at present only such factors as floor space, light, ventilation, convenience, and kind of equipment, whether standard or otherwise, in good or bad repair, and whether it was adequate to meet the requirements of tests as specified.

It may be said in passing that highway materials laboratories seem to be set up in any place obtainable provided the space is not needed for other purposes. Basements seem to be a favorite selection, but top floors or attic floors are nearly as popular. Such locations may be made quite comfortable, but with much difficulty. In one instance, it was observed that the basement was so damp as to prohibit the storage of cement there for more than a few hours. In another case steam pipes, although asbestos covered, increased the temperature in winter until it became almost unbearable except when ventilating fans were in operation. This same condition is also found on top floors under the roof when the sun beats down upon it.

The problem of obtaining adequate light presents itself to a large majority of laboratories. Electric lights are usually available, but there are some operations which should be done in good daylight in order to obtain best results. A frequent mistake, observed in laboratories occupying remodeled office buildings, was that of attempting to make the light from a few windows cover more than the space for which they were intended.

This brings up the point of personal efficiency which is a big item in testing as well as any other kind of work. Excessive moisture or heat will rapidly lower the vitality of an operator and should he be engaged in a series of comparative tests which depend on uniform manipulation, a likely source of trouble is easily recognized. Either insufficient or glaring light is tiresome to the eyes and the probable error in reading instruments and performing other close work increases accordingly.

Much may be done toward increasing efficiency in a laboratory by judiciously arranging the equipment. Of course, there is a limit to the amount of material which can be made to occupy a given space, but it is true of several laboratories that more careful planning would have greatly facilitated the work. Besides the inconvenience of operating in crowded quarters there is a decided disadvantage in not being able to inspect, clean, and repair machinery in the proper manner. With one exception, and that a small laboratory, it was observed that the congested equipment was in more or less poor condition.

The rock-testing equipment, it was found, usually received less care and attention than any other kind of apparatus. Sieves and screens came next, and last but of equal importance the water storage for cement mortar specimens was too often slighted. The principal result of negligence of rock-testing apparatus is the accumulation of dust and corrosion. Sieves uncared for become clogged with sand particles, or if improperly cleaned the wires are damaged to such an extent that the sieves soon become unfit for use. Occasionally sieves were seen which had corroded from contact with air in a chemical laboratory until they had become totally unfit for use for sand or cement testing.

MUCH FAULTY STORAGE OF CEMENT SPECIMENS

In many cases it was noted that certain details of the specifications of the American Society for Testing Materials for storage of cement specimens were not being followed. This applies particularly to the temperature of the clean water in the storage tanks. It is realized by everyone that to maintain the temperature at 70° F. at all times is difficult, perhaps practically impossible in some places. Nevertheless the specifications should be adhered to as closely as possible because variations from them present a likely source of error in results. Contrast the condition found in one laboratory where a large but shallow pan of water containing briquets is placed on the laboratory table with a small Bunsen flame under it to keep it from freezing and that of another laboratory which has a constant-temperature moist room in which are kept the storage tanks. A simple and rather effective arrangement to overcome this difficulty is to have a continuous flow of water from a supply into which is inserted an electrical heating element. By regulating the flow of water the temperature may be maintained at approximately 70° F. and at the same time the running water keeps the tanks clean. The laboratory of the Bureau of Public Roads has recently installed such a system, the details of which will be gladly furnished upon application.

One other obvious fault with the equipment of a number of laboratories was the lack of essential apparatus or the use of makeshifts of very doubtful efficiency. A surprising number of instances were found in which a moist closet consisting of wet cloths laid over fresh specimens was considered quite satisfactory. Rubber gloves, although advised by standard specifications, were thought to be superfluous. In one instance the quality of rock was determined with absolutely no equipment other than a hammer and United States Department of Agriculture Bulletin 348. Such are the results of inadequate funds, congested quarters, improper interpretation of specifications, and frequently lack of confidence of administrators in the testing engineers.

Briefly then we can say that more thought for the welfare of laboratory operators, more careful arrangement of apparatus, and a wiser selection of equipment would undoubtedly increase the efficiency of the average road materials laboratory many times over.

It may be of interest to review briefly some of the peculiarities observed in methods of testing. In all cases the methods of testing actually employed were compared with the methods recommended by the committee on tests and investigations of the American Association of State Highway Officials as approved for Federal-aid work by the Bureau of Public Roads.[1] Wherever possible, operators were observed while actually engaged in routine testing. Additional information was obtained by conversation with the operators and by general observation. As a result numerous variations from standard methods were noted which may be of interest in showing in a general way just how closely the standards are being followed by the men who do the work.

COMMON MISTAKES IN TESTING CEMENT

Although all Portland cement is supposed to be tested under the specification of the American Society for Testing Materials, which has remained practically unchanged since 1916, more variations in methods were noted in testing this material than any other. They ranged all the way from distinct violations of specification requirements, caused either by inexperience or prejudice, to minor variations in technique due to the personal equation of the operator, or to differences of opinion regarding the proper interpretation of the specifications. As would be expected, errors in testing were more serious in the newer laboratories than in those which had become well established. In some of the older commercial laboratories, however, operators were found who were apparently testing cement more nearly in accordance with methods in vogue in the early nineties than in accordance with present standards. It was more difficult to convince some of these men that their methods were incorrect than the young, inexperienced operators. Although errors in conducting practically all of the tests of Portland cement were observed, by far the greatest departure from standard methods occurred in connection with the preparation and storage of briquets for the standard tension test.

One or two illustrations will serve to call attention to conditions which, although fortunately not common, actually did exist at the time of inspection. In one case an operator was observed mixing a standard mortar in an ordinary iron pan with a trowel, and storing the briquets under a moist cloth for the night. In more than one instance operators were in the habit of ramming the standard mortar into the mold with the handle of a trowel. One man when questioned regarding this practice replied that it was the only way he could get the cement to pass the specifications. In still another case an operator was attempting to determine the normal consistency of a neat cement paste by mixing with the bare hands on a soft wooden surface. Quite a large number of testers were found to be disregarding altogether the specific requirement in the American Society for Testing Materials standards that the molds must be filled without ramming. This was particularly noticeable among the older men, some of whom evidently believed that they could get more consistent results by putting a lot of work into this operation. To sum up, it seemed to be a matter of general observation that, whereas the methods of testing cement employed by the newer organizations were usually such as to yield results lower than normal, the work of most of the commercial and some of the older State and university laboratories would have, as a rule, the opposite effect. Taken as a whole, conditions would certainly justify a vigorous campaign of education among cement testing laboratories on the part of those interested, and would seem to justify to a certain extent the contention of the cement manufacturers that unreliable test data are being turned out in some cases.

VARIATIONS IN TESTING OF AGGREGATES LESS SERIOUS

Variations from standard methods of testing concrete aggregates were not so common nor so serious as in the case of cement. In some instances operators did not seem to have a definite idea of the effect of maximum size of aggregate on the accuracy of the mechanical analysis. In other cases, the effect of size of sample on the efficiency of a mechanical shaker was no appreciated. For instance, one operator was found who was expecting the mechanical shaker to sieve accurately a 1,000-gram sample of sand in 10 minutes. With

[1] See U. S. Dept. Agr. Bull. 1216.

regard to the mortar-strength test for sand a rather serious error was discovered in quite a few cases. This was the tendency on the part of some operators to gage the sand mortar with the same amount of water in all cases, regardless of the character or grading of the sand being tested. Anyone familiar with tests of this nature knows to what such a practice might lead. Elutriation tests of fine aggregates were apparently being conducted in a satisfactory manner except that there appeared to be no agreement in regard to the size of sample to be used. The color test for organic matter was being carried out correctly as far as could be observed. In the abrasion test of gravel aggregates, however, a condition was noted to which attention should be called. This was the practice of some laboratories of making this test indiscriminately on uncrushed or crushed gravel, or a combination of the two. Experience has shown that wide variations in results are obtained when no attention is paid to the amount of crushed fragments in a test charge.

TESTING OF ROCK AND CONCRETE

Apparently the only rock test made to any great extent is the standard Deval abrasion test. The principal variations in methods of conducting this test were found in the preparation of the sample. In some cases great care was taken to secure pieces of uniform size and shape, while in others the sample was merely broken up by hand until the requisite number of pieces had been secured. Lack of attention to details of this nature undoubtedly explains some of the discrepancies in the results of tests which are continually appearing, and which unfortunately often lead the highway engineer to the conclusion that the tests themselves are basically wrong. Although many of the laboratories visited were equipped to make complete rock tests, many instances were encountered where routine tests other than abrasion were seldom if ever made, due principally to the difficulty experienced in the manufacture and use of rock drills. Wherever such tests were made, however, they seemed to be in accordance with standard practice.

Numerous variations were noted in methods of testing concrete in the laboratory. This was probably due largely to the fact that until recently there has been no uniform practice by which laboratories could be guided. The recent tentative specifications of the American Society for Testing Materials, however, have made it possible for all laboratories to follow methods sufficiently uniform to secure comparable results. One detail in connection with the crushing strength tests for concrete which has been overlooked by a number of laboratories should be mentioned. This is the matter of the proper capping of test specimens. The present practice requires that specimens be capped with either neat Portland cement or a mixture of Portland cement and plaster of Paris. The majority of the laboratories inspected, however, were still using the old plaster capping, which has been found to give considerably lower results especially on long-time specimens.

The writers do not wish to be understood as saying all laboratories visited were conducting tests of materials incorrectly. Most of the older State and university testing laboratories, as well as some of the commercial laboratories were found entirely satisfactory insofar as actual methods of testing were concerned. The one outstanding exception to this rule was the matter of cement testing which, as has been pointed out, was found to be in a most unsatisfactory state. It is safe to say, however, that in the majority of the laboratories inspected, incorrect methods of testing of a more or less serious nature were being followed. The writers may be pardoned therefore for emphasizing again the necessity for some action which will lead to an improvement along this line, especially as there is every reason to believe that the conditions noted are representative of the country as a whole.

PERSONNEL STANDARDS NEED REVISION UPWARD

One of the most difficult of laboratory problems is that of personnel. This is especially true of the operators comparatively far down in the salary scale, because it is this class which is most subject to change and therefore most uncertain. Certain observations and comments upon the character of personnel employed in the testing of road materials as evidenced by this inspection follow:

In the majority of cases the laboratory head or supervisor was found to be a competent engineer capable of appreciating the importance of his work and its relation to the other branches of the highway department. In some of the newer State laboratories, however, the men in charge were found to be greatly handicapped by lack of experience. This is probably due to the fact that when new State highway laboratories are established it is the usual custom to put an engineer already in the organization in charge instead of an experienced testing engineer. Such a practice works out satisfactorily in the end provided a man is chosen who has had enough laboratory training to enable him to see the situation from the point of view of the testing engineer as well as of that of the highway engineer.

Among the university laboratories a tendency was noted for the man in charge to give too little time to the actual supervision of the work of the operators, a condition which is usually brought about by requiring him to devote the major portion of his time to teaching. This practice should be discouraged. In a modern highway testing laboratory there is certainly enough for the testing engineer to do without requiring him to spend most of his time in the class room. The constant overlapping of highway and university work is one of the principal arguments against the use of university laboratories for the routine testing of materials.

The assistants employed in routine testing of materials, that is, the actual operators, fall into two general classes: Permanent operators; and temporary assistants, usually students. In those State laboratories where it had been found possible to build up a permanent organization the operators appeared to be as reliable and dependable as could be expected in view of the low scale of pay which obtained in most cases. In these organizations it is the usual custom to work on research problems during the winter months when routine work is at a minimum, and in this way keep a permanent force busy throughout the year. This is, of course, an ideal arrangement and is the goal towards which every road material testing laboratory should aim, since it not only makes it possible to build up a permanent organization but utilizes the facilities of the laboratory to the fullest extent.

The employment of students to conduct routine tests, which is the practice in some of the university

laboratories, should be discouraged. In the first place, they do not, as a rule, remain long enough in the laboratory to become proficient in testing work. In the second place, their interest in the work of the laboratory is secondary to their university work, and being temporary employees they have not the incentive to continued effort which characterizes a permanent man, anxious to advance. Of course, there were exceptions, but as a rule, it may be said that more actual errors in testing were observed on the part of students than any other class of operators.

With regard to the general question of laboratory personnel the writers take exception very strenuously to the belief that anyone can make routine tests. If there is one job in connection with highway work which requires preliminary training and experience it is that of the routine operator in the testing laboratory. The disposition of thousands of dollars worth of materials often depends upon the results of a single test, and yet frequently we find a 16 or 17 year old youth doing the work for a salary which is less than that of the typist who transcribes his report.

THE RELATION BETWEEN THE LABORATORY AND THE JOB

State highway testing laboratories as at present organized may be roughly divided into two general classes: Those which are a coordinate branch of the highway department, the head of the laboratory having complete supervision over the testing and inspection of all materials; and those which make tests only when required by the construction division, the chief having no jurisdiction over the ultimate disposition of materials. In the first case, the laboratory is usually part of a general Bureau of Inspections and Tests whose activities extend into the field in the form of materials inspection at the point of production or on the job. Such bureaus also frequently maintain a staff of prospectors who, working in cooperation with State geologists, are constantly developing new sources of supply. Such organizations are ideal but so far are found in only a handful of States. Examples of materials organizations developed along the above lines are those of the Pennsylvania, Illinois, New Jersey, Iowa, Minnesota, North Carolina, Washington, and Utah State Highway Departments. That the highway executives of these States believe the development of the materials testing side of highway engineering pays is evidenced by the size and importance of the materials testing organizations in these States. It is suggested that any State highway official who may be contemplating the expansion of the department of material inspection and tests, would do well to study the methods used in some of these State highway departments.

State highway laboratories of the second class are more numerous. In some cases they are doing good work; in most cases they are doing work which is as good as could be expected with the funds available. They are all handicapped by the absence of that intimate contact with the job which is so essential if the testing department is to function to its fullest extent. The writers believe that the testing or "materials" engineer rather than the highway engineer is the logical person to assume responsibility for the quality of the materials of construction. In the first place, he is in a better position to interpret intelligently the results of laboratory tests because he realizes the limitations to which the tests are subject. He is therefore better qualified to determine what tolerances should be allowed in specific cases. In the second place, he is interested solely in materials, whereas to the highway engineer, materials are but one of his many troubles. The testing engineer should, on the other hand, be in constant touch with actual construction, not only so that he may see how the materials he has tested are working out in practice but also in order that he may learn the point of view of the construction engineer. In other words he should be actually a "materials" engineer, a man equally familiar with both the methods of testing materials and their use in highway construction.

For reasons which have been pointed out the writers are not in sympathy with the general idea of using university laboratories for routine control testing of highway materials. Such laboratories may be and are utilized for carrying on research and many of them are doing excellent work along this line at the present time. The routine control testing should, however, be done by a laboratory under the sole jurisdiction of the highway department with a competent materials engineer in charge who is responsible directly to the chief engineer for all materials used.

Many of the State highway departments in the past have depended entirely on commercial laboratories for control testing of such materials as Portland cement, paving brick, bituminous aggregates, etc. There can be no objection whatever to this practice provided the work is done by an established firm with a reputation for thorough inspection. The great expansion in road-building within the last few years has resulted, however, in the organization of many new commercial laboratories, advertised as fully equipped to handle all kinds of road materials inspections. They are being established principally in those States which have the poorest facilities for doing the work themselves. It was possible for the writers to visit a few of these laboratories and though the time at their disposal was not sufficient to make a thorough study possible, enough was learned to indicate that some of the laboratories were far from being satisfactorily equipped or organized for highway materials testing.

(Continued from page 13)

of this the standard strength of solution now used in the laboratory is 1 per cent. Apart from the fact that this concentration is almost as destructive as much higher ones, its use has the added advantage of more nearly approaching ordinary bad field conditions.

As will be noted, the general form of the curve in Figure 2 based on the data of Table 1 is quite similar to that in Figure 3 based on compression test data of Table 2, although the difference between the 1 per cent and 4 per cent points is somewhat greater on the curve in Figure 3 than in Figure 2.

ROAD MATERIAL TESTS AND INSPECTION NEWS

STANDARDIZATION OF THE CRUSHING STRENGTH TEST FOR ROCK

Working in cooperation with the Road Materials Committee of the American Society for Testing Materials, the Bureau of Public Roads has been conducting a series of tests with a view to determining the best size and shape of test specimen for compression tests of rock. Although this test is used quite frequently in connection with the examination of stone for railroad ballast and building construction, and to a lesser degree, in connection with road construction, the method of conducting the test has never been standardized. The data now available are not comparable because many of the older tests were made on cubical specimens of various dimensions, whereas most of the later tests have been made on cylinders either 1 inch or 2 inches in diameter and of various heights. This particular study is a part of a general investigation looking to the standardization of the crushing strength test by the American Society for Testing Materials. The investigation will ultimately include not only a study of the size and shape of the test piece, but also the effect of other factors which influence the test results, such as the rate of application of the load and the character of the bearing surfaces of the test piece.

The first series of tests made in connection with the study of shape and size of test piece has been completed. In this series two samples of granite and a sample of limestone were tested. Cylinders both 1 inch and 2 inches in diameter were prepared, with heights of 1, 1½, and 2 inches for the 1-inch size, and 2, 3, and 4 inches for the 2-inch size. In making the tests, the other two factors which may affect the results were kept constant. The rate of application of the load was approximately 0.1 inch per minute, while the bearing surfaces of all specimens were ground plane with carborundum and water on a grinding lap and tested directly without the use of any capping whatever. The bureau has found that much higher unit stresses are obtained on stone when tested in this manner than when either a cement or plaster cap is used.

The results of these tests are shown in the following table. Each figure represents the average crushing strength obtained from three tests.

Dimensions of cylinder		Crushing strength		
Diameter	Height	No. 1 granite	No. 2 granite	Limestone
Inches	*Inches*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
1	1	32,000	23,500	6,000
1	1½	31,500	20,500	5,500
1	2	32,500	22,500	5,000
2	2	35,500	29,500	9,000
2	3	33,000	26,000	8,500
2	4	31,000	29,000	8,000

A rather interesting feature of the tests on the granite is that the lowest crushing strength, as a rule, was obtained with specimens, the height of which was one and one-half times their diameter. Tests made by other members of the committee seem to substantiate the values here shown. Tests on the limestone, however, show very little variation in result due to height of cylinder, although the values obtained from the cylinders 2 inches in diameter are somewhat higher than those obtained on cylinders 1 inch in diameter. This, it will be noted, is also true of the two samples of granite tested.

PROPORTIONING CONCRETE FOR TESTING PURPOSES

It is advisable to proportion materials for laboratory concrete specimens by weight. This insures uniformity not so easily obtained volumetrically due to variations in compaction.

In the laboratory of the Bureau of Public Roads, the materials for a concrete specimen are measured as follows:

The weight per cubic foot of the coarse aggregate and fine aggregate is determined according to the method described in United States Department of Agriculture Bulletin 949. A volume of 5,000 cubic centimeters of coarse aggregate furnishes enough concrete for one 6 by 12 inch cylinder.

By proportion

$$\frac{62.37}{W_a} = \frac{V_w}{W}$$

or

$$W = \frac{V_w \times W_a}{62.37}$$

in which

W = weight in grams of volume of aggregate required.

V_w = weight in grams of water equivalent to volume of aggregate required.

W_a = weight in pounds of a cubic foot of aggregate.

62.37 = weight in pounds of a cubic foot of water.

Example—

Required: 1 : 2½ : 5 mix of concrete in 6 inch by 12 inch cylinder.

Data given: Weight of cement per cubic foot, 94 pounds; weight of sand per cubic foot, 110 pounds; weight of gravel per cubic foot, 104 pounds.

Computations: 1 : 2½ : 5 mix.
1000 : 2500 : 5000.

$$\text{Cement } \frac{1000 \times 94}{62.37} = 1{,}507 \text{ grams.}$$

$$\text{Sand } \frac{2500 \times 11}{62.37} = 4{,}410 \text{ grams.}$$

$$\text{Gravel } \frac{5000 \times 104}{62.37} = 8{,}340 \text{ grams.}$$

Proportions by weight: 1,507 : 4,410 : 8,340.

STATIC LOAD TESTS ON PAVEMENT SLABS

The Bureau of Public Roads is about to conduct static-load tests on two series of pavement slabs which were previously used to study the effect of impact. The slabs are 7 feet square and vary considerably in their construction. Bituminous concrete bases and tops, concrete bases and one-course concrete pave-

ments are represented. Thicknesses and mixes are also varied. One series of slabs rests on a dry clay subgrade; the other is supported by a clay subgrade kept constantly moist. The slab corners will be tested under static loads and corresponding values of deflection, stress in the slab, and the amount of load will be taken until failure occurs.

The load will be applied by means of a hydraulic jack resting on the slab and working against a large, cylindrical, steel tank filled with water. The tank is 30 feet long and 6 feet in diameter. It is estimated that it will weigh 30 tons when filled. Mounted horizontally on suitable supports it will be moved on rollers into position over the slabs.

After the slabs now in place have been tested, they will be replaced by new slabs for the purpose of studying the effect of static loads on several of the more modern designs of concrete pavements, also the extent to which various types of longitudinal center joints transfer the load from one side of the pavement to the other.

PORTABLE OUTFIT FOR TESTING AGGREGATES

The Bureau of Public Roads has prepared and distributed to its materials engineers a number of portable testing outfits for use in the field examination of aggregates. Each outfit consists of a set of 8 half-cylindrical sieves and screens, ranging in size of opening from 100-mesh to 1-inch round; a 30-pound spring scale for making mechanical analyses of coarse aggregate and other weight determinations; a 200-gram spring balance for making mechanical analyses of concrete and asphalt sands; a portable stove for drying sand; a geologist's hammer; and bottles for silt and colorimetric determinations on concrete sand, etc. The equipment is contained in a small carrying case 18 inches long and weighs, together with the case, about 15 pounds.

COMPRESSION TEST FOR CONCRETE

A series of tests was made recently by the Bureau of Public Roads in an effort to reduce to a minimum the variation in the characteristics of mortar compression specimens resulting from lack of uniformity in the molding method. The mortar compression test has been coming into greater favor recently for investigational purposes and no doubt would receive general approval as a standard strength test also, if the method were standardized so as to insure uniform results.

In the bureau's tests four operators worked independently but used the same equipment and apparatus. The series of tests included 7 and 28 day specimens of briquettes and 2 by 4 inch cylinders of 1:3 Ottawa sand mortar. Briquettes were made according to standard specifications. Cylinders were made according to A. S. T. M. Tentative Standard Specifications C9–16T. Three sets of cylinders were made, using three styles of tampers, the suggested A. S. T. M. form, a one-fourth-inch bullet-pointed rod and a five-eighths-inch rod with a rounded end. Twenty-five strokes of the tamper were used in each case. Each operator repeated the series on 3 different days.

Comparing the work of different operators there was less variation in the strength of compression specimens made with the five-eighths-inch rod, than in the specimens made with either of the other styles of tampers; but the one-fourth-inch rod gave results nearly as consistent.

An operator could duplicate his own results most easily with the A. S. T. M. tamper, the five-eighths-inch rod coming next in favor. More uniform results were obtained by the operators in the tension test, probably because most cement operators have had more experience with this test.

Variation between individual breaks of sets was practically the same in all cases except those molded with the one-fourth-inch rod, in which case the percentage variation was considerably higher.

THE CUSHIONING QUALITIES OF MOTOR TRUCK TIRES

The results of a large number of tests by the United States Bureau of Public Roads prove that the deflection of motor-truck tires under static load is not a reliable measure of their relative shock-absorbing qualities, especially when the tires compared are not of the same general type. When the tires compared are of the same type—that is, when solid rubber and cushion tires are compared with each other, or when two or more pneumatic tires are compared—the static-load comparison is a fair indication of their relative cushioning properties.

It is not a proper basis of comparison between solid rubber tires and pneumatic tires, probably for the reason that the deflection of the solid rubber tire is dependent largely upon the shape of the tire and the quality of the rubber, and that the resistance to deformation is not proportional to the loads applied, but increases at a greater rate than the increment of loading. The deflection of pneumatic tires with respect to the loading is practically uniform and proportional to the loads. The reistance to the loading is only slightly dependent upon the material of the tire, and is mainly dependent upon the compressed air within. Although the rubber composition of some solid tires is apparently softer than a pneumatic tire as a whole, it will be seen that the cushioning properties of the pneumatic tire are much more effective than those of the solid or a cushion tire. The cushioning properties of the pneumatic tire remain in proportion to the load and still function at the end of the blow. This is not true with regard to the action of solid rubber tires. In these the rubber increases in density under increasing loads and deforms in such a manner that the unit intensity of loading at the area of contact is very greatly increased, thus losing in cushioning value and presenting greater resistance to the load or blow as it is brought to rest, with the result that the impact may be very greatly increased.

The cushioning effect of the various tires is clearly shown by the tests made by the bureau. As typical of the relative characteristics of the several types, consider a 3½-ton truck, using different tires, and carrying a load of 4½ tons, the testing speed in all cases being 15 miles per hour. This truck was allowed to strike an obstruction 2 inches high and 4 inches wide lying flat on the pavement. The same truck was also subjected to a drop test in which it was caused to drop 2 inches from one level to another. No change was made in the truck, except that three different types of tires were used. At one rear wheel the sprung weight of this truck was 5,300 pounds, the unsprung weight 1,700 pounds, and the total load was 7,000 pounds. With worn solid tires the impact force as measured was 26,500 pounds; with new solid tires, 18,700 pounds; and with pneumatic tires, 8,000 pounds, when the truck was run over the 2-inch obstruction. When the same truck was operated in the 2-inch drop test, the impact force developed when using worn solid tires was 23,500 pounds; new solid tires, 17,000 pounds; and pneumatic tires, 8,200 pounds. The results show the remarkable cushioning effect of pneumatic tires.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MAY 31, 1924

STATES	FISCAL YEARS 1917-1923			FISCAL YEAR 1924									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1923			PROJECTS COMPLETED SINCE JUNE 30, 1923			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 3,789,011.74	$ 1,794,703.21	367.9	$ 809,709.89	$ 391,544.33	96.2	$ 14,343,932.92	$ 7,111,741.36	928.0	$	$		$ 412,921.54	Alabama
Arizona	6,518,025.52	3,195,689.68	372.1	1,920,339.99	1,091,994.20	155.7	2,419,995.90	1,439,148.62	164.7	282,404.40	172,577.32	32.8	543,287.62	Arizona
Arkansas	8,623,205.72	3,309,572.12	736.2	2,376,036.54	1,074,497.36	201.1	6,206,905.23	1,973,663.06	334.9	1,663,527.73	646,234.70	96.1	900,535.69	Arkansas
California	9,495,547.60	3,538,967.90	340.6	4,503,527.43	2,108,130.27	193.2	11,670,617.36	6,277,613.82	430.2	207,198.09	122,246.87	3.9	2,681,306.36	California
Colorado	6,470,695.56	3,136,238.67	426.6	1,564,560.31	859,240.49	74.1	4,240,501.64	2,300,955.98	161.1	309,349.90	152,128.34	9.6	1,750,935.56	Colorado
Connecticut	2,568,643.95	1,026,148.95	60.7	128,610.43	64,306.21	3.8	3,411,072.42	1,209,614.44	61.4				606,612.59	Connecticut
Delaware	2,196,610.22	621,154.83	45.2	960,222.00	386,560.00	27.3	836,095.80	321,022.50	22.9	76,669.90	31,961.10	0.2	13,206.40	Delaware
Florida	69,466.31	29,700.63	15.6	604,477.52	288,175.17	23.3	8,578,162.96	4,232,785.06	255.5	1,037,974.30	440,221.40	13.7	408,668.22	Florida
Georgia	13,960,499.47	6,363,703.60	925.5	3,141,534.04	1,559,525.28	231.3	8,553,999.69	4,110,539.70	721.5	812,013.98	356,385.91	60.4	76,719.62	Georgia
Idaho	6,890,673.73	3,272,111.01	426.1	1,301,024.19	920,284.51	80.7	1,263,572.26	745,053.76	90.1	751,242.99	422,890.76	53.9	480,693.95	Idaho
Illinois	25,211,526.23	11,414,291.96	743.6	1,596,036.76	786,682.95	56.1	14,427,978.03	7,026,599.49	488.5	2,598,414.58	603,256.08	87.2	401,793.64	Illinois
Indiana	5,513,487.76	2,668,277.55	157.0	1,945,251.03	928,993.26	64.6	15,173,499.83	7,426,710.72	504.3	967,026.91	464,350.92	39.5	984,156.23	Indiana
Iowa	16,040,079.62	6,056,136.49	1,069.4	7,155,698.57	3,180,995.38	623.5	7,282,691.38	3,361,757.08	569.1	1,372,229.60	647,600.00	65.3	11,499.73	Iowa
Kansas	12,216,428.90	4,249,438.34	349.7	4,646,077.17	1,707,714.99	142.2	16,043,026.16	5,993,105.03	563.4	2,572,909.95	1,064,206.84	110.5	204,593.76	Kansas
Kentucky	6,620,289.90	2,789,991.16	286.0	4,202,690.51	1,823,956.13	154.7	7,573,552.00	3,663,928.55	329.3	101,092.14	50,541.07	4.0	631,737.65	Kentucky
Louisiana	7,131,394.90	3,157,128.03	590.4	1,234,851.99	427,311.82	61.1	5,557,561.95	2,674,116.23	370.0				11,585.03	Louisiana
Maine	4,662,954.19	2,242,963.03	159.8	2,248,204.59	1,056,972.35	70.9	1,272,125.82	633,936.15	49.2	804,168.95	399,778.95	34.6	69,969.16	Maine
Maryland	5,309,394.94	2,524,843.54	185.0	1,450,649.49	688,478.24	59.2	1,943,430.49	666,562.25	55.3	820,658.35	60,000.00	25.1	73,120.96	Maryland
Massachusetts	6,941,975.16	2,802,957.88	163.7	3,221,789.48	1,251,509.34	66.5	4,316,018.48	1,555,087.16	77.7	914,893.53	291,788.94	14.4	928,630.46	Massachusetts
Michigan	9,529,041.53	4,343,817.14	363.4	3,621,638.91	1,584,197.90	121.3	13,230,787.70	6,480,284.08	477.3	201,747.87	93,445.50	6.1	1,151,202.65	Michigan
Minnesota	19,928,671.37	8,004,277.87	1,893.4	4,208,889.87	1,891,565.20	398.6	7,511,903.19	3,092,454.43	551.5	385,121.03	134,500.00	52.7	84,714.94	Minnesota
Mississippi	4,746,771.31	2,284,557.86	462.4	2,763,348.19	1,355,346.34	169.9	8,046,353.57	4,015,246.28	505.6	427,290.23	190,310.42	17.9	391,440.45	Mississippi
Missouri	5,272,757.09	2,358,848.60	365.6	5,982,354.23	2,783,592.39	427.6	13,892,214.29	6,732,072.94	576.4	2,788,890.47	1,273,290.95	149.0	2,363,997.37	Missouri
Montana	7,935,185.30	3,896,053.50	711.5	824,817.33	431,454.03	64.0	2,144,062.85	1,530,753.64	229.0	977,724.46	512,114.80	108.5	3,051,557.94	Montana
Nebraska	3,159,814.47	1,449,675.04	475.6	4,597,022.49	2,205,282.17	939.7	6,132,216.62	2,963,295.74	641.0	1,716,789.48	853,214.77	168.0	2,402,322.94	Nebraska
Nevada	3,049,109.29	1,562,111.35	203.6	411,136.23	291,513.63	22.0	4,392,421.46	3,734,897.69	354.0	48,297.53	42,125.10	0.2	312,049.98	Nevada
New Hampshire	2,336,997.00	1,129,004.69	140.5	739,753.19	358,862.89	30.8	710,627.11	330,403.37	24.8	239,231.68	109,995.68	7.2	141,072.36	New Hampshire
New Jersey	5,423,362.96	1,975,783.52	112.1	2,200,432.16	685,747.97	36.6	9,936,391.60	2,625,213.16	64.6	417,573.68	107,667.49	41.5	259,421.93	New Jersey
New Mexico	4,706,909.12	2,433,663.68	617.1	599,377.33	325,186.00	97.2	5,315,301.97	3,423,018.97	604.0	1,547,940.33	1,047,545.43	102.7	174,389.04	New Mexico
New York	9,926,633.82	4,358,659.70	290.6	7,915,755.70	3,364,013.15	250.2	24,370,626.96	9,507,456.47	560.3	5,033,900.00	1,740,295.00	105.5	4,074,617.82	New York
North Carolina	11,770,363.76	5,325,763.37	926.8	797,369.21	350,994.29	57.9	9,136,546.91	3,284,593.17	246.9	2,842,006.66	1,259,959.16	74.9	375,694.86	North Carolina
North Dakota	5,142,157.55	2,492,599.09	792.5	3,946,815.56	1,925,906.33	795.4	2,440,363.59	1,194,816.41	400.2	853,212.51	426,606.69	113.2	1,145,019.36	North Dakota
Ohio	22,840,050.09	7,966,325.17	662.8	9,934,698.45	3,773,542.82	286.9	12,491,044.13	4,730,089.66	347.6	1,561,096.50	499,108.47	42.8	376,293.19	Ohio
Oklahoma	9,795,634.54	4,417,789.46	371.5	3,191,230.72	1,471,063.57	125.8	7,903,091.38	3,774,328.47	351.6	2,566,768.77	1,090,617.34	111.8	29,715.46	Oklahoma
Oregon	10,391,362.48	4,782,736.38	534.1	1,691,510.69	1,036,357.41	121.5	2,900,425.31	1,491,878.83	161.5				28,356.23	Oregon
Pennsylvania	33,090,029.45	12,615,058.78	652.8	3,346,501.08	1,379,436.00	70.8	8,797,632.19	2,816,365.00	151.9	12,645,174.35	3,498,242.50	225.3	926,557.50	Pennsylvania
Rhode Island	1,494,075.35	647,634.96	38.6	290,321.90	131,593.00	7.4	1,087,787.06	411,759.49	22.7	334,265.25	99,147.31	5.6	297,291.37	Rhode Island
South Carolina	6,191,485.41	2,895,754.37	655.1	1,892,361.00	769,071.67	251.3	6,753,180.11	2,673,767.50	514.7	648,631.57	203,553.71	36.4	101,370.58	South Carolina
South Dakota	4,758,907.24	2,320,007.33	534.6	3,915,690.62	1,924,629.94	455.3	5,825,817.64	2,964,990.42	716.5	149,283.79	74,641.88	18.4	225,267.25	South Dakota
Tennessee	3,210,970.94	1,535,762.18	112.5	2,084,853.17	1,019,402.48	97.6	14,010,617.93	6,809,832.94	455.8	2,386,464.89	919,212.07	72.1	111,686.46	Tennessee
Texas	29,172,501.06	11,094,069.77	2259.6	11,846,295.22	4,542,975.45	901.4	25,850,956.09	10,276,130.76	1633.8	3,824,285.38	1,399,550.26	214.1	1,317.00	Texas
Utah	2,175,007.41	1,196,437.68	116.6	1,129,416.34	699,369.24	102.4	3,926,766.16	2,496,027.94	279.5	918,151.77	596,199.59	53.3	280,697.75	Utah
Vermont	1,370,715.63	669,433.75	53.1	551,399.53	273,335.37	21.3	1,257,846.96	603,240.34	37.8	517,413.80	253,242.99	9.7	369,101.28	Vermont
Virginia	6,823,978.36	3,279,029.56	402.6	3,061,720.63	1,447,951.63	155.0	8,504,603.59	3,910,842.83	309.8	1,177,475.06	110,693.00	36.6	395,873.43	Virginia
Washington	10,356,980.82	4,819,105.70	419.8	646,289.24	287,467.07	35.4	3,217,995.26	1,497,800.00	125.1	349,409.83	151,900.00	13.0	27,097.06	Washington
West Virginia	4,100,225.73	1,800,932.41	206.3	1,389,522.22	564,109.12	49.3	4,833,696.80	2,136,265.06	163.8	619,277.45	275,200.22	21.7	179,349.73	West Virginia
Wisconsin	15,504,691.39	5,996,989.24	1077.6	3,249,211.76	1,444,044.33	247.7	2,665,045.55	1,323,212.92	114.9	252,613.91	126,306.06	13.7	2,910,298.13	Wisconsin
Wyoming	4,601,439.36	2,210,774.20	534.8	1,438,619.13	815,095.43	137.8	4,089,496.50	2,576,868.47	301.2	325,739.27	109,591.52	41.3	39,649.25	Wyoming
TOTALS	$ 407,704,641.14	$ 174,044,673.82	23297.2	$ 132,919,640.92	$ 59,623,915.30	9801.5	$ 356,240,287.50	$ 162,110,847.62	16999.6	$ 60,947,241.80	$ 23,114,437.00	2518.4	$ 33,106,126.26	TOTALS.

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 53,556,879.31 Federal aid $ 24,333,867.02 Miles 2,441.9

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale ot a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

REPORTS

Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.
Report of the Chief of the Bureau of Public Roads for 1920.
Report of the Chief of the Bureau of Public Roads for 1921.
*Report of the Chief of the Bureau of Public Roads for 1922. 5c.
*Report of the Chief of the Bureau of Public Roads for 1923. 5c.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
*389. Public Road Mileage and Revenues in the Central, Mountain, and Pacific States, 1914. 15c.
390. Public Road Mileage in the United States, 1914. A Summary.
*393. Economic Surveys of County Highway Improvement. 35c.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*555. Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials. 10c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 15c.
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Where do we Go from Here?

WE HAVE MADE SOME PROGRESS along the way toward complete improvement of the highways of this country in the last 10 years. The statistics give us credit for more than 400,000 miles of surfaced road, and nearly all of it has been improved within the decade. Perhaps it is time to ask the doughboy's question, Where do we go from here?

ONE THING IS CERTAIN, and that is that we are going to have better roads in this country. The public demands them. There has been manifest of late some reaction against the expenditure of public funds for roads. There can be no avoiding that expenditure. With more than 15,000,000 motor vehicles now operating on our highways, additional mileage of improved highways and the maintenance of those already improved is a huge work that can neither be laid aside nor neglected. The cost of operating and maintaining these motor vehicles on the highways in their present condition will be greater than the cost of improving and maintaining the roadways so that they will be equal to the traffic demands.

IN OTHER WORDS, with the number of motor vehicles now in operation, the public at large will pay in other ways a sum greater than the cost of improved highways if we do not improve the highways. Unless the States build them the locality must, and the voice of experience proclaims it a very foolish thing for the people of a State to decide not to support an adequate program of State road building, as a means of decreasing taxes, because it is much more likely to increase taxes.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 5 ▽ JULY, 1924

THE MOTOR TRUCK IS NOW THE MOST IMPORTANT AGENCY TRANSPORTING MILK FROM PRODUCER TO DISTRIBUTOR

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 5, NO. 5 JULY, 1924 H. S. FAIRBANK, Editor

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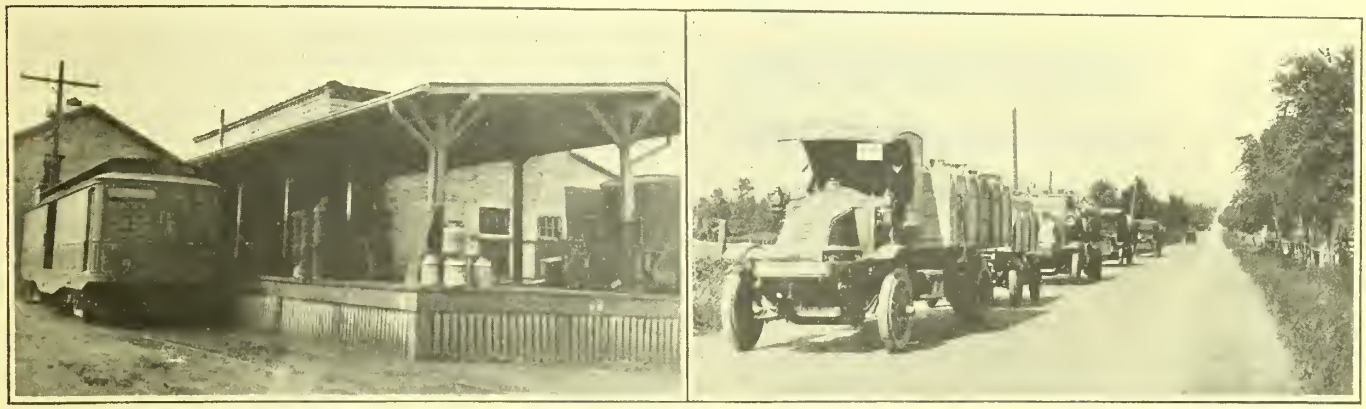
TRANSPORTATION OF MILK BY MOTOR TRUCK

RESULTS OF A SURVEY OF MILK TRANSPORTATION FOR EIGHT LARGE CITIES

BY HENRY R. TRUMBOWER, ECONOMIST, U. S. BUREAU OF PUBLIC ROADS [1]



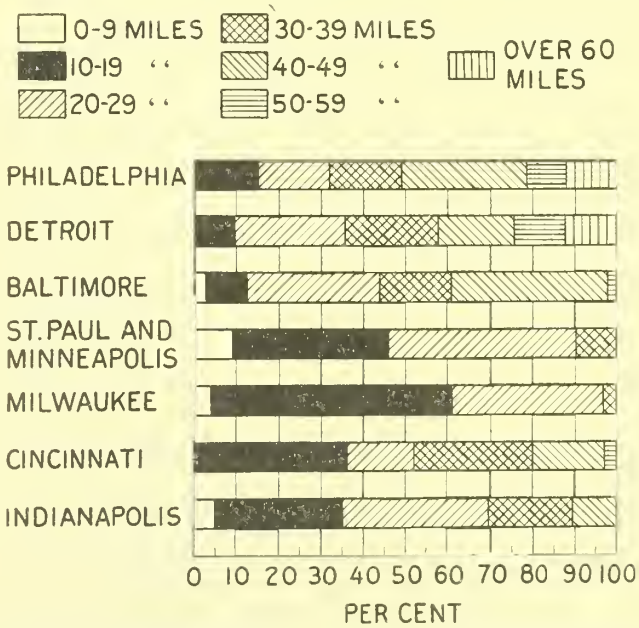
Delivery of milk by motor truck has greatly increased in volume in recent years. Direct movement from producer to city distributor is one of the principal advantages of this method of transportation

IN THE early days milk was transported but short distances from the farms where it was produced to the consumers living in towns and cities. The area in which a city's milk supply was produced, known as its milk shed, was naturally restricted as the horse and wagon was the customary means of bringing the milk into the city. Railroad shipments of milk did not begin until about 1838 when Boston began to be supplied by this new means of transportation.[2] These railroad shipments began in almost all cases by the carrying of a few cans of milk on the front platform of a baggage car.

The increase in the population of the larger cities was such that the farming territory from which the milk could be hauled by horse and wagon was no longer capable of furnishing all of the supply demanded by these large centers of population. Milk was still brought in over the country roads from near-by farms, but the larger portion of the supply came by rail from more distant production areas. After the advent of electric interurban railroads these were used in many cases to bring in milk which had formerly been hauled by the steam railroads or milk which had been taken to the city by wagon.

HIGHWAY TRANSPORTATION NOW THE MOST IMPORTANT AGENCY FOR MILK DELIVERY TO LARGE CITIES

The use of the motor truck over improved highways has brought about a complete change in conditions. Except for the largest cities of the country the motor truck has now become the important agency in transporting milk from the producers to the city distributors and retailers. In order to determine to what extent the motor truck is used in bringing milk into the cities over the highways a survey and investigation of the transportation of milk by motor truck has been made covering this situation in a number of representative cities. The cities selected are Baltimore, Philadelphia, Cincinnati, Detroit, Milwaukee, St. Paul, Minneapolis, and Indianapolis. In all of these cities except Baltimore and Philadelphia, motor trucks transport approximately 90 per cent of all the milk received. Philadelphia, which is the largest of the cities surveyed, received only 20 per cent of its milk by motor truck. The proportion of Philadelphia's total supply which is shipped in by motor truck has been increasing in the last few years, but the percentage is low because the city, on account of its heavy demands, draws its supply from such a large area in which the shipping distances become too great for motor truck hauling. Milwaukee and Cincinnati obtain almost their whole milk supply by motor truck.



Milk trucks in various areas classified according to mileage zones in which they operate

[1] The field work on which this report is based was performed by E. L. Browne, Agricultural Economist, U. S. Bureau of Public Roads.

[2] Parker, H. N., City Milk Supply, 1917.

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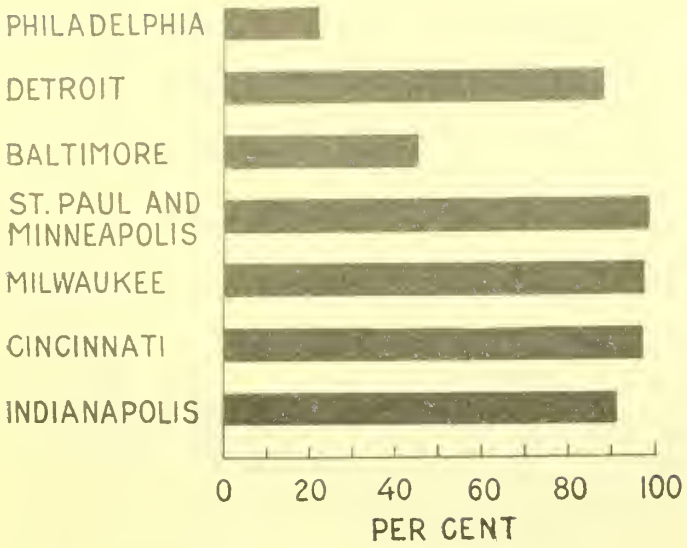


Percentage of total milk supply of eight cities delivered by motor truck

SUMMARY OF PRACTICE IN MILK TRANSPORTATION

For the eight cities studied a total of 633 motor trucks were engaged in the transportation of milk; 65.6 per cent of these trucks operated within the 0–29 mile zone; 27.6 per cent in the 30–49 mile zone; and the rest, 6.8 per cent, traveled routes 50 miles and over in length. A large part of this movement is over comparatively short distances. In a number of cities the use of the motor truck has tended to draw in the boundaries of the city's milk shed and has stimulated the dairy industry in sections closer to the city because of the more convenient method of transportation.

The motor trucks engaged in this business are for the most part of the smaller capacity type; the 1 to 2 ton trucks constitute 57.1 per cent of the total number. The trucks over 4-ton capacity amount to only 7.3 per cent. Detroit is the only city which shows a very large development in the use of the tank truck. This type of equipment is used only in connection with the transportation of milk from receiving or collecting plants in the country to the city dairies and is usually operated over fairly long routes. It is not adapted for use in connection with pick-up movements.

There is a lack of uniformity in the rates charged by the motor trucks for this service. In each city there are independent truck operators charging different rates for hauling the same distance. When a rate comparison is attempted for the several cities even greater differentials are noted. In individual cases truck operators will charge higher rates where the whole route or a part of it is over poor or unimproved roads. This differential in most cases ranges between 5 and 6 cents per 100 pounds. For distances from 30 to 39 miles the motor truck rate ranges from 35 cents to 40 cents per 100 pounds, or the equivalent of about 20 cents per ton-mile. For shorter distances the rate is higher. It is the uniform practice for the rate charged to include the return movement of empty cans. In very few cases has any attempt been made by motor truck operators to develop return loads of merchandise and other commodities destined for points in the country.

The operation of the motor trucks engaged in this traffic is almost wholly in the hands of individuals owning one or two trucks who live in rural sections where the milk is produced. Very few of the city dairy companies and city retailers have taken up the task of transporting milk from the country to the city. Only in cases where they operate receiving stations do they perform any of this hauling themselves or engage motor-truck transportation concerns to do it for them by contract.

Where railroad rates were compared with motor-truck rates it was usually found that the motor-truck rates were the same as the railroad rates or somewhat higher. Even though the truck rates are higher the tendency is to ship by motor trucks where the distances are not too great for trucks to operate. Use of the motor truck in shipping milk eliminates the city terminal transportation costs which have to be met where shipments are made by railroad. The milk producer is also saved the cost and time which is involved where he is required to take his milk to the rural railroad stations if he can ship directly from his farm by motor truck.

Cooperative transportation organizations have not been developed to any great extent in the areas surrounding cities studied in this investigation. The only motor trucks operated by farmers upon a cooperative basis were found in the Baltimore area where several organizations of this kind have entered this field and made a success of it.

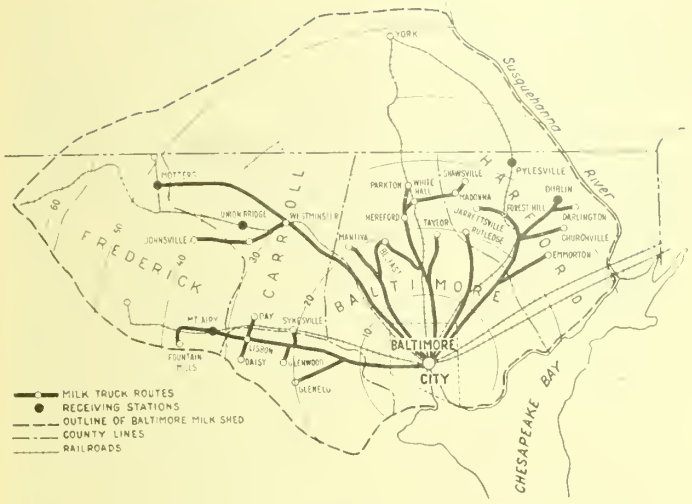
TRUCK TRANSPORTATION IN BALTIMORE AREA HAS INCREASED STEADILY SINCE 1915

In 1915, according to the records of the health department, Baltimore received approximately 25 per cent of its milk supply by wagons and motor trucks operating over the highways from producing centers close by, and 75 per cent was received over the lines of several railroads. Prior to that time trucks had not entered the field. The amount of milk brought into Baltimore by these several means of transportation for the period 1912–1923 is shown in the following tabulation:

Milk transportation into Baltimore by railroad, wagon, and motor truck for the years 1912–1923

Year	Railroad		Wagon		Motor truck		Total for year
	Amount for year	Per cent of total	Amount for year	Per cent of total	Amount for year	Per cent of total	
	Gallons		*Gallons*		*Gallons*		*Gallons*
1912	8,862,868	80	2,190,000	20			11,052,868
1913	9,345,860	81	2,190,000	19			11,535,860
1914	9,723,718	84	1,825,000	16			11,548,718
1915	9,525,707	76	2,190,000	17	751,990	7	12,467,697
1916	9,305,778	73	1,825,000	14	1,649,225	13	12,780,003
1917	8,990,850	72	1,825,000	14	1,748,847	14	12,564,697
1918	10,146,237	73	1,825,000	14	1,747,370	13	13,718,607
1919	10,866,692	70	1,825,000	12	2,798,540	18	15,490,232
1920	12,002,655	65	1,825,000	10	4,507,612	25	18,335,267
1921	12,219,462	68			5,573,325	32	17,792,787
1922	12,055,906	64			6,736,675	36	18,792,581
1923	10,940,107	55			8,944,285	45	19,884,392

The amount of milk brought in by wagon was of necessity estimated; but the motor-truck and railroad shipments appear to be accurately reported. The amount of milk shipped by railroad increased steadily until a maximum was reached in 1921; since then there has been a noticeable decline. Consumption of milk in the city has increased each year, reaching a total of 19,884,392 gallons in 1923. Of this amount 45 per cent was transported by motor truck from near-by producing points and 55 per cent was shipped by railroad. Since 1920 the milk brought in by wagon has



Map of Baltimore milk shed showing routes and mileage zones.

become a negligible quantity. In 1923 practically the same amount was shipped by railroad as in 1919 yet the consumption had increased by more than 4,000,000 gallons. The burden of supplying the increased consumption has been taken up by the motor truck. The tendency is for the motor truck to bring in a larger and larger percentage of the city's total milk supply.

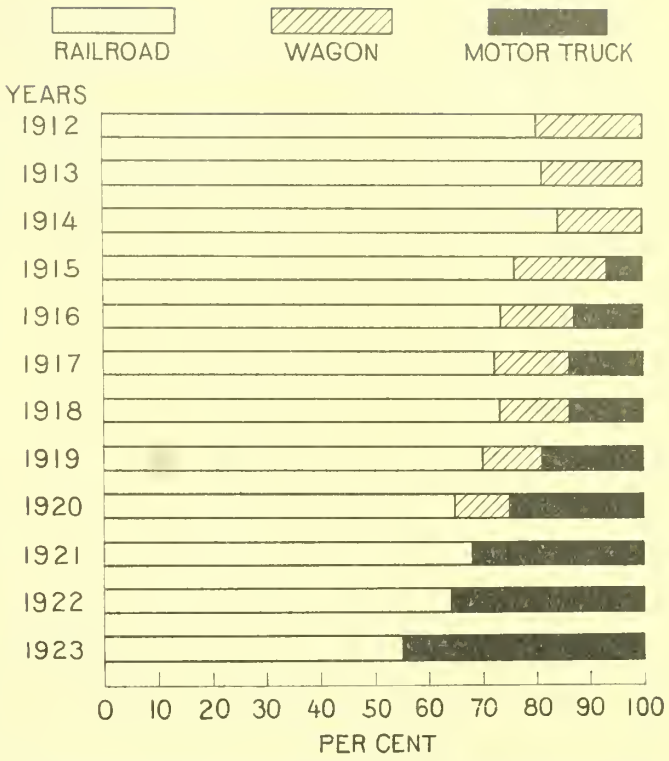
ZONES OF OPERATION AND KINDS OF TRUCKS USED

At the close of 1923 there were 41 motor trucks engaged in the transportation of milk which brought in an average total of 23,720 gallons of milk a day. The distances over which these trucks operated ranged from 6 miles to 53 miles. In the following table are shown the number of trucks and also the mileage zones.

Zone	Number of trucks	Per cent	Zone	Number of trucks	Per cent
Miles			*Miles*		
0–9	1	2.4	40–49	15	36.6
10–19	4	9.8	50–59	1	2.4
20–29	13	31.7	Total	41	100.0
30–39	7	17.1			

It is noteworthy that only five trucks, or about 12 per cent, come from points less than 20 miles away. Approximately 85 per cent of the trucks come from points 20 to 49 miles distant; only one truck has its point of origin over 50 miles distant. A study of the character of the agricultural land tributary to the Baltimore market indicates that the farm lands beyond the 20-mile zone lend themselves more readily to the dairy industry and the production of milk. This seems to account, at least partially, for the concentration of the industry in these more or less distant zones.

The motor trucks engaged in this kind of transportation are of varying capacities, ranging from 1 ton to 5 tons. There are one or two trucks of three-fourth-ton capacity used as feeder trucks in bringing milk in from side roads to the main highways where the milk is picked up by the larger trucks which run all the way into the city. A complete classification of trucks as to their capacities is shown in the following table.

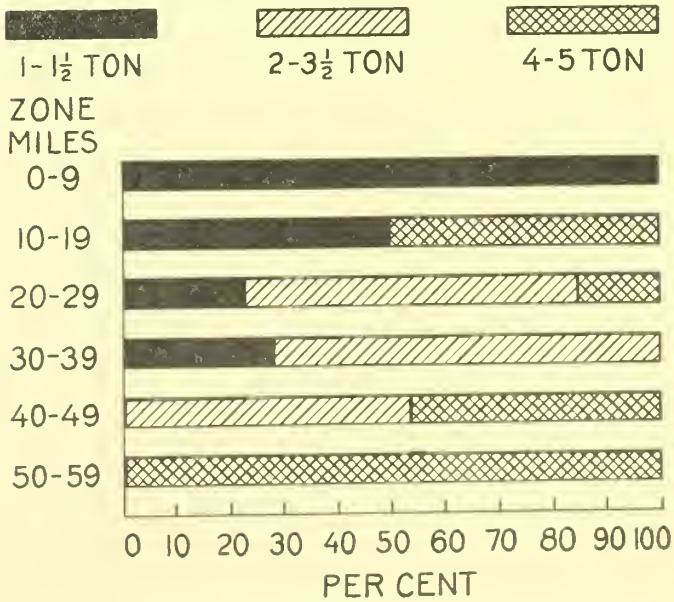


Percentage of milk transported into Baltimore by railroad, wagon, and motor truck, 1912–1923

Classification by capacity of trucks operating in Baltimore area

Capacity	Number	Per cent	Capacity	Number	Per cent
Tons			*Tons*		
1	3	7.3	3½	1	2.4
1½	5	12.2	4	4	9.8
2	5	12.2	5	8	19.6
2½	3	7.3			
3	12	29.2	Total	41	100.0

The largest number of trucks of any one capacity is found in the 3-ton class; 12 trucks compose this class or 29.2 per cent of the total number. Approximately



Percentage of motor trucks of different capacities in various mileage zones, Baltimore area

two-thirds of all the trucks are of 3-ton capacity and less. Heavy trucks of 4 and 5 ton capacity are not used as extensively in this type of hauling as trucks of lighter weight and capacity.

It is of interest to ascertain to what extent the trucks of large and of small capacities are used in the several mileage zones. In making this comparison the trucks are grouped into three classes; trucks from 1 to 1½ ton capacity, 2 to 3½ ton capacity, and 4 to 5 ton capacity.

Motor trucks of different capacities in various mileage zones—Baltimore

Mileage zone	1 to 1½ ton		2 to 3½ ton		4 to 5 ton	
	Number	Per cent	Number	Per cent	Number	Per cent
0–9	1	100				
10–19	2	50			2	50
20–29	3	23.1	8	61.6	2	15.3
30–39	2	28.6	5	71.4		
40–49			8	53.3	7	46.7
50–59					1	100.0

It is the tendency to use trucks of large capacity for the longer hauls. Of the trucks of 4 to 5 ton capacity, eight are used in the zones of 40 miles and over, while only four are used in the zones up to 30 miles. The trucks of 2 to 3½ ton capacity, of which there are the largest number, operate in the 20–50 mile zone. The trucks of smaller capacity are used only within the 40-mile zone. The trucks of larger capacity can ordinarily be operated over a greater distance more economically because there are certain items of expense, chief of which is labor, which are approximately the same regardless of the distance and the capacity of the truck.

COVERED TRUCKS USED TO MEET HEALTH DEPARTMENT RULES

Of the 41 trucks operating in the Baltimore area, 39 are equipped with solid tires; one truck is fully equipped with pneumatic tires and one has pneumatic tires on the front wheels. The types of bodies used on the milk trucks have largely changed from the open stake body to the van or enclosed type, principally on account of the requirements of the city health department. The rules of the health department provide that the milk cans shall not be exposed to the sun or dust in transit or while the milk is standing on the platforms awaiting the arrival of the truck. While a complete covering of the cans in transit with a heavy canvas meets the requirements, nevertheless a majority of the trucks are of the inclosed type. Out of the 41 motor trucks engaged in this kind of hauling, 30 are of the van type, 8 have open bodies, and 3 are tank trucks. The fact that a considerable number of the trucks engaged in the transportation of milk are also used in the return load movement is another reason for the preponderance of the van type of truck. Where various kinds of merchandise make up the return load, a truck with an inclosed body has advantages over a truck with an open or a stake body.

The health department's order also requires covered loading platforms in the country where the trucks receive the milk from the farmers. These platforms are built at convenient intervals along the highway covered by the truck routes. Where it is possible one platform is built to accommodate several producers, thereby eliminating the too-frequent stopping of the truck. Trucks were at first loaded from the rear by backing up to the loading platforms and dropping down the end gate. This method was satisfactory where there was sufficient space to back the truck around, but difficulties were encountered on narrow highways. Recently the idea of a side gate or a side door in the truck body has been developed and is rapidly gaining favor with motor-truck operators because of the greater ease and saving of time in loading, and also because of the greater ease in unloading in the narrow streets and alleys adjoining the city milk plants. The use of side gates also facilitates the handling of the empty cans in connection with a return load of merchandise; the empty cans can be distributed more easily along the route without disturbing or rearranging the return load of freight. The empty cans are piled on their sides the full height of the truck at one end of the body, leaving the maximum amount of room for freight. Some of the trucks have the side gates on the right side only; others have them on both sides. Icing of milk hauled by motor trucks in the summer months has never been practiced to any great extent, although a 1½-ton truck iced during the entire summer of 1923 brought very gratifying results. The bacteria count was greatly reduced and practically no sour milk had to be returned.

At the present time only three tank trucks are operated regularly in the Baltimore territory. These are used by two large city dairies in hauling milk from several of their receiving stations in the country to their city plants. Glass-lined tanks of 1,000-gallon capacity are mounted on 3-ton and 5-ton chasses. The use of this class of equipment is limited to the transporting of milk from country receiving stations to city plants. In the Baltimore area most of the milk shipped by motor truck is sent directly from the producer to the city distributor, which accounts for the fact that the tank truck is not used to a greater extent.

FOUR CLASSES OF OPERATORS ENGAGED IN MILK TRANSPORTATION

The owners and operators of trucks engaged in the transportation of milk can be divided into four classes. First, those individuals and companies which operate their trucks as common carriers under permits secured from the public service commission; second, milk producers organized as cooperative associations for the purpose of transporting and marketing their milk; third, individuals who buy milk from producers and transport it and sell it to city dealers; fourth, retail dealers and dairies who collect milk directly or through receiving stations from producers and distribute it through their own organizations. This classification is shown in the following table both as to the number of owners and operators comprising each group and the number of trucks in operation.

Number of operators in each class of ownership and number of trucks operated in each class, Baltimore area

Group	Operators		Trucks	
	Number	Per cent	Number	Per cent
Public service	11	48.0	20	48.7
Cooperative	7	30.5	13	31.8
Individuals	1	4.2	3	7.3
Dairies	4	17.3	5	12.2
Total	23	100.0	41	100.0

That 23 operators own the 41 trucks which are engaged in this kind of business shows that there is little, if any, concentration of ownership. The largest number of trucks operated by one concern is five, which are all listed as common-carrier trucks. Approxi-

mately half of the trucks are operated under permits issued by the public service commission, which restricts them to definite routes and schedules and makes the owners file their rates and charges, which are subject to public regulation. There are seven cooperative organizations operating 13 milk trucks. Under the laws of the State these trucks have been exempt from the provisions of the public service regulation statute, but recently this has been amended so that even cooperative associations will have to obtain permits and will have to pay the license fees which are exacted from these types of carriers. Only four dairies are engaged in hauling or bringing in milk which they use in their plants. It is rather significant that the retail dealers take little part in the business of transporting milk to their plants. This is largely accounted for by the fact that the movement of milk from receiving stations is limited in quantity.

A number of cooperative associations have been in existence for some time. One of them dates back to 1917, when it was organized for the purpose of hauling milk by motor truck to Baltimore. The rates charged the stockholders are usually the same that commercial-truck operators charge for similar service. Some of the associations, which were organized with sufficient capital, are able to pay dividends to their stockholders in addition to setting aside sufficient funds to replace worn out and retired equipment. Other associations were organized with small amounts of capital and secured the money for the purchase of the original equipment largely through loans. These associations use the balance of their earnings over and above operating expenses to pay off their indebtedness and to add to their equipment. There is one cooperative association which now possesses rolling stock valued at about $20,000, most of which was purchased with surplus earnings.

COMPARISON OF MOTOR-TRUCK AND RAILROAD RATES IN BALTIMORE AREA

The rates charged for transporting milk conform fairly closely to the rates charged by the railroads. As a matter of fact the rail rates were the basis for making charges for truck hauling when the motor truck first entered this business. No case was found where the truck rate was less than the railroad rate, a great many cases were found where the railroad rate and the truck rate were the same for like distances, and in some instances it was found that the truck rates exceeded the rail rates. The railroad rates are proportional to the length of the haul to a greater degree than the truck rates. The railroad tariffs frequently name a different rate for each shipping point along a certain line, while the truck rates are largely of the "blanket" type. The blanket rate is also frequently found in railroad rates as is seen in the table below in which comparisons of railroad and motor-truck rates are made for like mileage zones. The railroad rates are taken from the milk tariffs of the Western Maryland Railroad which carries approximately 57 per cent of the railroad shipments and 29 per cent of the total milk shipments into Baltimore. This railroad serves the territory lying to the north and northwest of Baltimore. The truck rates are typical rates from points in the several mileage zones in that same general territory.

According to the table the railroad rate is "blanketed" from 20 to 50 miles at 3 cents. In this zone the truck rates vary from 3 cents to 4.5 cents per gallon. In the 15–19 mile zone the railroad rate is 2.5 cents, whereas the truck rate is 3 to 3.5 cents. In this zone the trucks are serving milk shippers who are not closely situated to railroads and are not readily served by them.

Comparative railroad and motor-truck rates for like mileage zones, Baltimore area

Railroad shipping point	Truck shipping point	Distance to Baltimore	Rate per gallon	
			Railroad	Truck
		Miles	*Cents*	*Cents*
Glyndon		15–19	2.5	
	Hypsie's Store			3.0
	Taylor			[1] 3.0–3.5
	Marble Hill			3.0
Falls		20–24	3.0	
	Hereford			3.0
	Sykesville			3.0
	Belfast			3.0
	Rutledge			3.5
	Lutherville			4.0
	Tracey's Shop			4.0
Patapsco		25–29	3.0	
	Lisbon			3.0
	Eldersburg			3.0
	Parkton			4.0
Westminster		30–34	3.0	
	Westminster			3.0
	Glenwood			3.0
	Emmorton			3.0
	Mount Airy			3.0
	Day			3.0
	Forest Hill			4.0
Wakefield		35–39	3.0	
	Churchville			3.0
New Windsor		40–44	3.0	
	Belfast Church			3.0
	Darlington			3.5
	Fountain Mills			4.0
	Shawsville			4.0
Union Bridge		45–49	3.0	
	Johnsville			[1] 4.0–4.5

[1] Oct. 1 to Mar. 31.

While certain truck rates shown in the table appear to be out of line with other truck rates in the same zone, an investigation has revealed that some of these truck routes follow rather poor roads which are at times difficult to travel, thus accounting for the higher rates. In two instances the rates for the winter months are higher than the rates for the summer solely because of bad road conditions. This shows in a very substantial way the value of improved roads to the farmer. Practically a truck load of milk is collected along 5 miles of unimproved highway where for a six months' period 3.5 cents is charged for transporting milk and for the rest of the year only 3 cents. The farmers and milk producers are required to pay $5 a day additional transportation charges on 1,000 gallons of milk because of the inferior type of highway. This amounts to $900 for the six months' period. Based on this actual outlay, the farmers could afford to make road improvements to the extent of $3,600 a mile if no other traffic were benefited at all besides this one truck load of milk.

The milk tariffs of the Baltimore & Ohio Railroad are fixed according to distance to a much greater degree than the Western Maryland. The following is taken from its tariff:

Baltimore & Ohio Railroad milk rates to Baltimore

From—	Distance	Rate per 10-gallon can
	Miles	*Cents*
Woodstock	21.7	27.0
Henryton	25.2	28.0
Gaither	30.1	29.5
Waterville	36.6	30.0
Bartholow	43.1	31.0
Monrovia	46.7	32.5
Ijamsville	50.2	34.0

ADVANTAGES OFFERED BY MOTOR-TRUCK TRANSPORTATION

In comparing motor-truck rates with railroad rates one must not lose sight of the fact that the motor-truck rates include pick-up service in many cases and in all cases delivery service at the city milk plant. When shipping by railroad farmers have to haul their milk varying distances from the farm to the station and the dairy or city milk dealer is obliged to haul the milk shipped by railroad from the city terminal to his manufacturing or distributing plant. The dealers in Baltimore estimate that it costs them from 0.5 cent to 1 cent per gallon for this terminal hauling which is entirely eliminated where the milk is brought directly to the plant from the points of production in the country. These costs naturally vary for different dealers, depending largely upon the distances they are removed from the railroad terminals. In Baltimore not one of the dairies is located directly on a railroad siding, so that in each case trucking, either by horses or by motor truck, is necessary to get the milk from the railroad car or platform to the plant.

The saving in time and expense to producers who ship by motor truck is shown by the following sample data which were secured from questionnaires filled out by milk producers shipping by truck and milk producers shipping by railroad. Forty-eight replies from each class of shippers were tabulated. Of the 48 shippers by truck, 23, or approximately 50 per cent, were located on truck routes and did not have to do any hauling. The remaining 25 hauled their milk to convenient points on truck routes a total of 22.43 miles, or each one hauled his milk an average of 0.86 mile. If these same 48 shippers who shipped by motor truck had all taken their milk to the nearest railroad stations they would have had to haul a total distance of 117.25 miles. The actual saving to all of these shippers was 94.82 miles or an average of 1.98 miles for each shipper.

The 48 shippers who shipped their milk by railroad hauled it a total distance of 61.45 miles or an average of 1.28 miles each. The average hauling on the part of the 48 shippers who shipped by truck was 0.47 mile each. The average saving in hauling distance which the truck shippers enjoyed over the same number of railroad shippers was 0.81 mile each.

It should be noted that this analysis shows also that the milk producers who are most likely to ship by truck are those who are more distantly removed from their respective railroad stations. Shippers who made use of the motor truck were on an average 2.44 miles removed from the railroad, whereas those who shipped by railroad were on the average 1.28 miles distant. To those shippers who are the greatest distance from the railroads and who live on or near roads which are adapted to truck routes the motor trucks bring the greatest benefits. Through the use of motor trucks operating over improved highways the dairy industry is promoted and developed in regions which hitherto were too far removed from railroads to be developed practically as producing areas for fluid milk destined for the city market.

PERSONAL SERVICE OF TRUCK OPERATORS A VALUABLE CONSIDERATION

One of the reasons given by milk producers for preferring to ship by motor truck rather than by railroad is the personal service which truck operators are able to give in looking after the interests of the shipper. It has been claimed that the percentage of cans lost in truck shipment is less than in railroad shipment on account of the direct daily return of cans. The 48 motor-truck shippers and 48 railroad shippers were asked for data with reference to these claims. The results of their answers are shown in the following tabulation:

Number of cans used and lost by railroad shippers and by motor truck shippers

Shippers	Average number of cans shipped daily	Total number of cans used by shippers	Total number of cans lost during year	Per cent of cans lost
48 truck shippers	225	517	19	3.6
48 railroad shippers	211	661	83	12.5

The railroad shipper must have a larger number of cans in use to ship approximately the same quantity of milk on account of the irregular return of empty milk cans by the railroad. A more significant fact is that the shipper by railroad suffers an annual loss of 12.5 per cent of his supply of cans, whereas the truck shipper loses only 3.6 per cent. The truck driver serves to a certain extent as the shipper's representative from the time the milk cans leave the farmer's gate or loading platform until the empty cans are returned later in the day. Responsibility is centered in one person for taking care of the shipper's property and the handling of the cans is limited to the one or more persons in charge of the truck. Shipments by railroad are handled more often and by a greater number of individuals who are employees of the railroad and the city dealers. Obviously the chances for losses are multiplied.

In order to give an idea of the time consumed in shipping milk by motor truck and the operations involved, there is presented below a day's log showing the operation of a representative motor truck engaged in the transportation of milk. The one-way distance covered is 27.5 miles. The truck after delivering its load of milk proceeded to a central freight depot for its return load.

Detailed log of a milk truck

Time	Miles	Operation	Elapsed time
6.30–7.30 m	8.0	Picking up milk	1 hour.
7.30–9.10 a. m	17.0	Run to city dairy	1 hour, 40 minutes.
9.10–9.22 a. m		Unloading milk	12 minutes.
9.22–9.40 a. m		Loading empty cans	18 minutes.
9.40–10 a. m	2.5	Run to freight depot	20 minutes.
10–11.30 a. m		Central freight depot, idle	1 hour, 30 minutes.
11.30–12 a. m		Loading freight	30 minutes.
12–12.30 p. m		Driver and helper at lunch	30 minutes.
12.30–2.30 p. m	19.5	Return run to first shipper	2 hours.
2 30–3.30 p. m	8.0	Unloading cans and freight	1 hour.
Total	55.0		9 hours.

The truck for which these data were obtained was a 4-ton truck of the van or inclosed-body type and carried a load of 120 cans of milk containing 1,200 gallons. The truck was away from its terminal garage a total of nine hours; six hours, or 66 per cent of the total time, were consumed by actual truck movement in picking up the load of milk from farmers along the route and in making the run to the city dairy plant, to the freight depot, and back again to the point of departure. In picking

up the milk 18 stops were made and that many shippers served. It should be observed that for only three hours, or 33 per cent of the operating time, the truck was not in motion and a part of this time was taken up in unloading the milk and getting back the empty cans and in obtaining a return load of freight.

The records of the dairy plants in Baltimore show that the motor trucks engaged in this business of transporting milk from the country to the city operate with a great degree of regularity. Snow removal on the Maryland highways is well taken care of, so that the main highways are seldom if ever closed on account of snow. Records are not available for all of the motor trucks as to regularity of operation. It was found, however, that for 24 trucks not a single trip had been missed in the years 1922 and 1923. This does not mean that each one of these individual trucks was operated daily. Emergency trucks may have been used at times, but no trip was missed on account of unfavorable weather conditions, and continuous and regular service was afforded to the shippers and to the public.

Flexibility of operating schedules is an advantage which motor-truck operators claim for their method of transportation. The time of departure in the morning is usually an hour later in the winter than in the summer months, which is a great convenience to the shippers. The earlier operation in the summer months makes it possible for the milk to be brought to the city before the heat of the day sets in.

REVENUE AND OPERATING COSTS AT BALTIMORE

Little data can be found relative to the revenues and expenses of individual operators, a condition which prevails quite generally in all kinds of motor trucking. From one of the city dairies information was obtained for 21 trucks, covering the yearly receipts, miles operated, loads carried, and truck capacities. This is given in the following table:

Revenues and operating data for 21 motor trucks, 1923

Truck capacity	Number of trucks	Average number gallons hauled per trip per truck	Average yearly mileage per truck	Average yearly revenues from milk per truck
		Gallons	*Miles*	
2 tons	2	387	16,425	$5,179
2½ tons	6	450	25,246	5,946
3 tons	5	620	23,600	6,716
3½ tons	5	830	24,820	8,869
4 tons	1	575	20,075	6,296
5 tons	2	913	18,980	9,991
Average		625	23,070	7,154

The average revenue received from the transportation of milk amounted to $7,154 per truck, or an average of 3.1 cents per gallon. The receipts of the 5-ton trucks were the largest, on account of the larger loads carried. In the case of the two 5-ton trucks the average yearly mileage per truck was less than in the case of most of the other trucks of smaller capacity. This furnishes the basis for computing the average receipts per ton-mile. The weight of the average load of 625 gallons, including cans, is 3.53 tons. The weight of the average load of returned cans is 0.6 tons. The number of ton-miles of both the milk and the empty cans is 47,639. In arriving at this figure it is assumed that the loads are carried for the whole distance, which is not exactly the case, inasmuch as the full load is not obtained at the very beginning of the trip. Without making any adjustment for this, we arrive at the average receipts per ton-mile of 15 cents.

For one of these trucks it was possible to obtain complete data covering operating expenses and revenues. This information is for a 5-ton, solid-tire truck operating on a milk route 22 miles one way over an improved highway. The total receipts were $10,014.40, consisting of $8,485 from the transportation of milk and $1,529.40 derived from return loads of freight and merchandise. The total operating expenses were $7,636.75, leaving $2,377.65 as the year's profit. It should be noted that fully 65 per cent of these profits were derived from return-load earnings. It appears therefore, highly desirable that this part of the business be developed to the fullest extent in order to have this type of transportation yield the maximum returns.

The year's operating expenses are fully segregated and analyzed in the following statement.

Cost analysis of year's operation of a 5-ton motor truck

Item	
1. Total investment (chassis, body, freight)	$5, 884. 00
2. Tire cost, 36″ by 6″ and 40″ by 6″ solid	304. 50
3. Investment less tire cost (amount to be depreciated)	5, 579. 50
OVERHEAD COSTS	
4. Interest on total investment per year (6 per cent on item 1)	$353. 04
5. Taxes	147. 10
6. License	1, 026. 48
7. Insurance	588. 40
8. Garage	240. 00
9. Pay-roll costs	2, 616. 00
10. Total overhead	4, 971. 02
RUNNING COSTS PER MILE	
11. Depreciation	$0. 0732
12. Tires	. 0200
13. Oil	. 0035
14. Fuel	. 0400
15. Repairs and maintenance	. 0302
16. Total running cost	. 1669
PAY-ROLL COSTS	
17. Driver's wages per week	$27. 00
18. Driver's overtime	
19. Helper's wages per week	19. 00
20. Supervision per week	4. 3075
21. Weekly pay roll	50. 3075
22. Yearly pay roll, item 24 by item 21	2, 616. 00
23. Days operated per year	363
24. Weeks operated	52
25. Total overhead cost per day	$13. 70
26. Miles operated per day	44
27. Miles operated per year	15, 972
28. Total overhead cost per mile	$0. 311
29. Total overhead costs per mile	$0. 311
30. Total running costs per mile	. 1669
31. Total cost per mile	. 4779
TOTAL COSTS	
32. Total overhead costs	$4, 971. 02
33. Total running costs, item 16 by item 27	2, 665. 73
34. Total costs	7, 636. 75

DAILY OPERATION OF MILK TRUCKS FAVORABLE TO LOW COSTS

The milk trucks make a good showing when subjected to a cost analysis because of the daily operation throughout the year which tends to spread the fixed charges over a relatively large mileage. Next to the pay-roll costs the license fee in this case is the largest item of overhead costs amounting to $1,026.48. This particular truck is licensed as a common carrier and is obliged to pay fees which exceed the license fees which the ordinary truck has to pay.

According to the law in effect up to this time motor trucks operated as common carriers on designated routes and schedules are divided into three classes. Class X includes all those trucks weighing 3 tons and less including carrying capacity which are charged one-fifth cent per ton multiplied by the number of miles the truck is operated in a year over the public highways of the State. Class Y includes those trucks in the 3 to 6 ton class which are charged two-fifths cent per ton-mile. Class Z includes those trucks in the 7 to 10 ton class which are charged three-fifths cent per ton-mile. The truck covered by this cost analysis was in the Z class and paid a license fee of three-fifths cent per ton-mile. The same kind of truck operated not as a common carrier would have had to pay only $150 a year without any restrictions as to mileage or distance. This large differential in license fees tends to limit the number of motor trucks engaged in the transportation as common carriers and probably accounts for the rather large number of trucks cooperatively owned which are in this service because cooperatively owned trucks have been exempted from the provisions of the public service laws relating to motor trucking. The legislature in its last session amended the law so as to include cooperatively owned trucks in the definition of common-carrier trucks. This change will take effect next year. It should be noted in this connection that this truck, the operating costs of which have just been analyzed, paid over 10 per cent of its gross earnings in taxes and license fees.

Return loads collected at a central depot have proven profitable to truck operators in the Baltimore area. Such business has not been developed extensively in other sections

One of the cooperative associations published a report of its financial operations for the year which is summarized as follows:

Revenue from milk transportation	$15,032.91
Revenue from other freight	6,069.70
Total	21,102.61
Operating expenses, taxes, and depreciation	18,597.37
Net revenues	2,505.24

This organization operated two trucks. Attention should again be called to the large revenues which were derived from freight hauled back into the country from the city. Had it not been for these earnings the association could not have shown any net revenues at the end of the year. The importance of obtaining return loads is recognized by most of the motor-truck operators. The revenue derived from the return load is regarded as just that much additional income which involves little or no additional costs.

In order to secure return loads a number of motor-truck concerns are maintaining jointly a central freight depot in the central part of the city's business section. Any freight or merchandise which is to be shipped to points and individuals along the different milk routes is concentrated at this depot. After the truck has unloaded its load of milk at the city dairy it swings around to the central depot to ascertain what freight is on hand for country points along its route.

CENTRAL DEPOT SOLVES RETURN LOAD PROBLEM

The central-depot idea appears to be a great success according to all the truck operators who make use of it. It does away with the hit-and-miss system of collecting merchandise all over the city from many business and manufacturing concerns. Such concerns maintain a city-delivery system of their own and can just as conveniently make deliveries at the central depot where the merchandise in turn is picked up by the milk trucks which are about to proceed on their return journey.

The larger trucks are generally operated by two men; one acting as the driver and the other as the helper. In the following table is shown the number of trucks operated by one man and by two according to truck capacity:

Number of men employed in operating trucks of various capacities

Truck capacity	Number of trucks employing 1 man	Number of trucks employing 2 men	Total number of men employed	Truck capacity	Number of trucks employing 1 man	Number of trucks employing 2 men	Total number of men employed
Tons				*Tons*			
1	3		3	3½		1	2
1½	5		5	4		4	8
2	5		5	5	2	6	14
2½	2	1	4				
3	2	10	22	Total	19	22	63

On the 3-ton trucks and trucks of larger capacities it is the practice to employ two men. In the above tabulation there are two 5-ton trucks in operation with only one man. Both of these are tank trucks which do not need the extra man.

The average hauling rate of approximately 3 cents per gallon is in the last analysis only a small part of what the city consumer pays for his milk. In December, 1923, the retail price of milk in Baltimore was 14 cents per quart, or 56 cents per gallon, delivered to his house in bottles. Of this amount the producer received 28 cents, the producer's association 1 cent as a brokerage charge, the city dealer 24 cents and the motor-truck operator 3 cents. The dealer's margin consists of 42.8 per cent of the retail price, the milk-truck operator receives 5.4 per cent, the producer's association 1.8 per cent and the farmer who produces the milk 50 per cent of what the consumer pays for his milk.

MILK SUPPLY TRANSPORTED BY MOTOR TRUCK INCREASING ANNUALLY IN PHILADELPHIA

Although the largest part of the milk received in Philadelphia is shipped by railroad the amount which is brought in by motor truck has shown a considerable increase in the last few years. Up to 1921 steam railroads and the electric railroads carried approximately 95 per cent of the city's total milk supply. In 1921 the amount brought in by motor truck showed substantial gains. The 1923 records show that a little over 20 per cent of the city's milk supply was received by motor truck. The detailed shipments are given in the following table:

Amount of milk shipped by rail and motor truck, Philadelphia area

	1921		1922		1923	
	Amount	Per cent	Amount	Per cent	Amount	Per cent
	Gallons		*Gallons*		*Gallons*	
Steam railroad	53,159,943	88.2	53,856,060	84.0	55,273,056	78.5
Electric railroad	1,382,517	2.2	1,190,911	1.9	922,501	1.3
Motor truck	5,778,285	9.6	9,015,026	14.1	14,162,289	20.2
Total	60,320,745	100.0	64,061,997	100.0	70,357,846	100.0

During this period there was a substantial increase in the amount of milk received. The motor truck took care of the greatest part of this increased supply although the steam railroads showed an increase in the amount which they handled. On the other hand the amount shipped over the electric interurban lines showed a falling off. In 1923 the motor trucks transported into the city slightly over 20 per cent of the total supply whereas less than 10 per cent was brought in by that means in 1921. The steam railroads' share of this traffic was 78.5 per cent in 1923 compared with 88.2 per cent in 1921. The investigation showed that the motor trucks took care of that part of the city's milk supply which was taken away from the electric interurban lines. The large volume of milk consumed in Philadelphia requires a milk shed extensive in area. A part of this area is so distant from the city that motor trucks can not successfully compete with the railroads for the business. Shipments are made by producers in New York, Pennsylvania, Maryland, Delaware, and New Jersey. The same situation prevails here as in other large cities such as New York and Boston. Although large quantities of milk are produced within a radius of 50 or 60 miles, all of this production does not find its way to the Philadelphia market because a large number of cities and populous communities derive their supplies of milk from the same territory.

LARGE-CAPACITY TRUCKS NUMEROUS

The number of motor trucks delivering milk in the city of Philadelphia and the distances over which they operate are given in the following table:

Number of trucks operating in various mileage zones, Philadelphia area

Mileage zones	Number of trucks	Per cent	Mileage zones	Number of trucks	Per cent
0–9	0	0	60–69	3	6
10–19	7	15	70–79	2	4
20–29	8	17	80–89	1	2
30–39	8	17		47	100
40–49	14	30			
50–59	4	9			

It is to be noted that of 47 trucks engaged in this business over 50 per cent operate over routes 40 miles and over in length. Thirty per cent of the trucks operate in the 40–49 mile zone. A truck starting at Churchville, Md., 81 miles distant, travels the greatest distance. This truck, at the time the survey was made, had been operating only a short time. It leaves Churchville at 9 a. m. and picks up its load of milk and arrives at the city dairy at about 4.30 p. m. and does not get back to its point of origin until about midnight. It is not believed that this operation can continue any length of time; at least, difficulties will be encountered in the summer months when heavy icing will be required. The long hours may also make two sets of drivers necessary. The two trucks which operate in the 70–79 mile zone leave the city in the morning and go out to receiving stations for their loads, and get back late in the afternoon.

The capacity of the motor trucks engaged in these operations is shown in the table below.

Classification of trucks by capacity, Philadelphia area

Capacity in tons	Number	Per cent	Capacity in tons	Number	Per cent
2	2	4	5	29	62
3	11	23	7½	1	2
3½	4	9		47	100

Trucks of small capacity are lacking. Only two trucks of the 2-ton type are found. The 5-ton truck predominates, 62 per cent of the total number belonging to this class. In the following table it is shown that the trucks of larger capacity operate in the longer mileage zones:

Number and capacity of trucks operating in various mileage zones, Philadelphia area

Mileage zone	2-ton	3-ton	3½-ton	5-ton	7½-ton	Total
0–9						
10–19	2	1		4		7
20–29		2	4	2		8
30–39		3		5		8
40–49		4		10		14
50–59				3	1	4
60–69				3		3
70–79				2		2
80–89		1				1
Total	2	11	4	29	1	47

RECEIVING STATIONS MAKE LARGE TRUCKS PRACTICABLE

The trucks of 3½-ton capacity and less do not operate over routes more than 50 miles in length. Over half of the 5-ton trucks operate from points more than 40 miles distant. The fact that a large part of the milk transported by motor truck into Philadelphia is milk which is collected at receiving stations and then hauled by truck directly to the city plants accounts partially for the larger trucks used and also for the greater distances. These receiving stations are operated by the dairies. To them the farmers bring their milk by motor truck or by wagon just as they haul it to the railroad stations or creameries. There are, however, a few receiving stations operated by dairy companies which collect the milk from the producers. The milk collected at these receiving stations comes from within a radius of 10 miles of the station. In the Philadelphia milk shed 15 receiving stations and 3 bottling stations are operated, from which the milk is transported by motor truck to the city distributing plants.

Out of the 47 trucks delivering milk to the city 26 haul directly from receiving stations and bottling plants and 21 pick up their loads from the producers. The average amount of milk transported from the receiving stations is 25,140 gallons per day. This is an average load of 967 gallons per truck. The 21 trucks engaged in hauling milk directly from the producer to the city plants transport 13,580 gallons per day or an average load of 647 gallons per truck. The milk transported from the receiving stations is all hauled by trucking concerns which have contracts with the dairies or by the dairy companies themselves. The large number of trucks of large capacity in this territory is to a great extent explained by the fact that such a large portion of the milk is transported from these receiving stations where full loads are available and where the carrying of large loads appears to be the most economical way of doing it.

CLASSIFICATION OF MILK TRUCKS ACCORDING TO OWNERSHIP

In the Philadelphia territory no cooperative associations are found engaged in hauling milk for groups of producers, nor was any development of the return load found. Those engaged in the transportation of milk by motor truck may be divided into three classes: First, the dairy companies which own their trucks and use them in hauling milk from the country receiving stations which they operate to their city plants; second, individuals or companies which enter into contracts to haul milk from the receiving stations to to the city plants (only one of these hauling contractors does any pick-up business); third, individuals who operate trucks from the country to the city plants and pick up the milk from producers who live along the routes over which they operate. The individual haulers all live and have their points of origin in the country. The hauling contractors and dairies go out into the country and bring in their supply of milk. In the following table is shown the number of those engaged in hauling milk divided into the above-mentioned classes and also the number and capacity of trucks operated by them.

Classification by capacity of truck operated by different classes of owners

Class	Number of trucks operated					
	2-ton	3-ton	3½-ton	5-ton	7½-ton	Total
Dairy companies			4	7		11
Hauling contractors				19	1	20
Individuals	2	11		3		16
	2	11	4	29	1	47

The dairy companies operated 11 trucks, or 23 per cent of the total number; the hauling contractors operated 20 trucks, or 43 per cent; and the individuals operated 16 trucks, or 34 per cent. It is significant that the trucks operated by the contractors are all 5-ton trucks with the exception of one which is of 7½-ton capacity.

IMPROVED ROADS MADE POSSIBLE A CONSIDERABLE SAVING IN MILK TRANSPORTATION

One of the largest dairies in Philadelphia, which has an average daily run of 22,000 gallons, receives all but 300 gallons by motor truck. Up to four years ago all this milk was shipped in by railroad. The superintendent of this plant stated that in 1920 when they first began to haul milk by motor truck the roads were in very bad shape, which tended to hinder this kind of operation. Since the roads have been improved more trucks have been added, so that almost all the shipments are by motor truck. This dairy now owns 12 glass-lined tanks which it furnishes to hauling contractors who mount them upon chasses owned by them. The dairy agrees to keep the tanks painted, but the truckman takes all responsibility in the operation of the truck. The dairy pays the contractor a flat rate of 30 cents per 100 pounds or approximately 2.6 cents per gallon for the transportation of milk in these tank trucks operated from the various receiving stations. The nearest one is 47 miles and the most distant one is 60 miles away. The dairy estimated that the savings resulting from this method of transportation compared with rail shipments from the same points and the hauling from city railroad terminal to the dairy amounted to 5 cents per 100 pounds. On a daily run of 22,000 gallons this means a daily saving of slightly over $90.

The average rate paid to individuals operating milk trucks from the country and to contractors for transporting milk in 10-gallon cans is approximately 35 cents per 100 pounds. This rate covers distances from 15 to 60 miles. A 35 cents per 100 pounds rate is about the same as a rate of 3 cents per gallon. The one truck which operates in the 80–89-mile zone receives 6 cents per gallon.

Delivering milk to a receiving station where it is cooled and run into a tank truck for transportation to the city

ALMOST ENTIRE CINCINNATI SUPPLY NOW DELIVERED BY MOTOR TRUCK

The year 1914 marked the beginning of the transportation of milk by motor truck to the Cincinnati market. By 1918 nearly 50 per cent of the daily shipments were by motor truck and the wagon shipments had practically ceased. The rest of the supply came in by railroad. In 1923 the year's shipments by motor truck were 12,430,000 gallons out of a total of 12,868,000 gallons received, or 97 per cent. In spite of the adverse road conditions and a decidedly hilly territory surrounding the city the motor truck shipments of milk increased in this 10-year period. Between the years 1915 and 1921 there was an increase of 36 per cent in the city's consumption of milk. It was mainly through the opening up of new producing territory through the use of the truck that the Cincinnati dealers and distributors were able to take care of the rapidly increasing consumption.

At the present time there is no "pick-up" milk brought in by the railroads; the only milk shipped by railroad is that brought from a receiving station at

Pierceville, Ind., a haul of 45 miles. It is predicted that the end of 1924 will see a complete motorization of Cincinnati's milk delivery when a new road will be completed leading from Pierceville to the Ohio State line joining a hard-surfaced road which will make it possible to ship the milk accumulated at the Pierceville station by truck to its destination. The extent to which Cincinnati receives its supply of milk by motor truck is significant when one takes into consideration the fact that there are four railroad lines running south from Cincinnati into Kentucky and nine steam railroad lines and seven traction lines radiating to the north, east, and west into Ohio and Indiana. The milk supply comes from approximately 3,200 producers; 60 per cent originates in Ohio, 30 per cent in Kentucky, and 10 per cent in Indiana. The area circumscribed by a radius of 30 miles supplies 80 per cent of the total amount of milk consumed and the remaining 20 per cent comes from the 30 to 60 mile zone.

LIGHT TRUCKS AND SHORT ROUTES NUMEROUS IN CINCINNATI AREA

The number of trucks operating in this area and the mileage zones in which they operate are shown in the following table.

Number and per cent of trucks in the various mileage zones, Cincinnati area

Mileage zone	Number of trucks	Per cent	Mile ge zone	Number of trucks	Per cent
0–9	0	0	40–49	15	17
10–19	32	36	50–59	3	3
20–29	15	16			
30–39	25	28		90	100

The fact that over a third of the trucks operate over routes of 20 miles and less is rather significant when compared with the situation in some of the other cities where data of this kind have been obtained.

The table below shows the number of trucks used in this territory classified according to capacity.

Number of trucks by capacities, Cincinnati area

Capacity in tons	Number of trucks	Per cent	Capacity in tons	Number of trucks	Per cent
1	16	18	3	2	3
1½	32	35	3½	12	13
2	18	20			
2½	10	11	Total	90	100

It is to be noted that over 50 per cent of the trucks are of 1-ton and 1½-ton capacities and that 3½-ton trucks are the largest operating in the territory. Small trucks are favored because the routes are not long and because the roads over which they have to operate contain a great many steep grades. All of the trucks are operated by one man to each truck.

The extent to which the trucks of smaller capacity are confined to the shorter routes is shown in the tabulation below.

Number of trucks operating in the various mileage zones by capacities

Mileage zone	1-ton	1½-ton	2-ton	2½-ton	3-ton	3½-ton	Total
0–9							
10–19	11	14	3	3		1	32
20–29	2	5	3	1	1	3	15
30–39	1	8	6	2	1	7	25
40–49	1	5	5	4			15
50–59	1		1			1	3
Total	16	32	18	10	2	12	90

While there are 48 trucks of 1-ton and 1½-ton capacity, 25 of these are operated in the 10–19 mile zone. Only four of the twelve 3½-ton trucks are in the 10–29 mile zone; the rest are operated over longer routes.

The type of body used very generally in this section is the ordinary rack body which can be used for both milk and stock hauling. Those trucks which are used exclusively for hauling milk are for the most part equipped with stake bodies. Livestock, particularly calves and hogs, are often transported along with a load of milk. The health department requires that where livestock is carried on the same truck with milk a light board partition of sufficient height be placed to separate entirely the livestock from the load of milk.

RETURN-LOAD BUSINESS NOT DEVELOPED

The glass-lined tank truck has never been tried. Most of the trucks are owned by individuals who live in the country and operate but one truck. The return-

Map of Cincinnati milk shed showing routes and mileage zones

load movement is not developed to any extent largely because no one has taken the initiative to organize and put in a central freight depot. One truck operator stated that he had tried to secure return loads but had to give it up because he found himself spending too much time driving from place to place in the city to gather his load. He felt that it paid him better to hurry home after delivering his load of milk and take a load of livestock back to the city in the afternoon. Such double trips could, of course, be made only where the route was of short distance. Several operators on routes from Kentucky have made arrangements to take back groceries and dry goods from wholesalers to country stores along their route. They felt they could give this kind of a return service because not much time was required of them to collect the return load but they refused to go into the general parcel and express business.

The number of operators and number of trucks operated by them is shown in the following table.

Number of truck operators by classes and number of trucks operated by each, Cincinnati area

Class	Operators		Trucks	
	Number	Per cent	Number	Per cent
Individuals	64	98.5	78	87
Dairies	1	1.5	12	13
Total	65	100.0	90	100

According to this statement the 90 trucks engaged in this traffic were owned by 65 operators. Only a few of the individuals owned and operated more than one truck. Only one dairy company which operated 12 trucks was directly engaged in the transportation of milk. All the milk hauled by its trucks was obtained from 9 receiving stations most of which were beyond the 30-mile limit.

RATES CHARGED IN CINCINNATI AREA

An examination of the map of the Cincinnati milk shed and the information collected in making this survey indicates that the recent extension of the milk shed has been largely into Indiana and Kentucky. The new milk routes which have been established tend to follow the new and improved roads. Road construction has made possible to many farmers a new industry with a market ready to absorb the output.

While there are no definite uniform rates in effect for the transportation of milk by motor truck it is found that motor-truck operators grade their charges somewhat in accordance with the distances hauled. These rates may be stated as follows:

Mileage zone	Rate in cents per 100 pounds	Approximate rate in cents per gallon	Mileage zone	Rate in cents per 100 pounds	Approximate rate in cents per gallon
10–19	25	2.1	40–49	40	3.4–3.8
20–29	30	2.5	50–59	50	4.3
30–39	35	3.0–3.4			

These rates approximate 1 cent per 100 pounds per mile. Variations are found where poor and difficult roads are encountered. There is also a growing tendency among the truck men to add a differential of 5 cents per 100 pounds during the winter months when their operating costs are higher and their loads smaller on account of the seasonal fluctuations in the production of milk.

The Ohio Legislature passed an act in 1923 which requires all motor trucks engaged in the transportation of goods for others to register and obtain permits as common carriers. This change affects the operating expenses of all of those engaged in the business of transporting milk. The act provides that common-carrier motor trucks pay, in addition to the regular license fees, certain common-carrier fees and that the operators of such common-carrier trucks must take out cargo, liability, and property damage insurance. A 2-ton truck, for example, with a 35-horsepower rating paid as regular license fees about $32 a year. Under this new law it will have to pay in addition $80 as a common-carrier fee and insurance premiums amounting to about $105. This legislation adds approximately $185 a year to the costs of operating this kind of a truck. It will be interesting to see what effect these increased costs will have upon the motor-truck rates and the number of trucks operated.

MOTOR TRUCK HAS BEEN THE MEANS OF PROVIDING FOR GREATLY INCREASED CONSUMPTION IN DETROIT

In 14 years, from 1910 to 1924, the population of Detroit grew from 465,766 to 1,250,000, an increase of 168 per cent. With the ever increasing demand for milk the Detroit distributors had to look for new producing areas. It is significant that the areas developed did not lie along the railroads and electric lines which had been furnishing the greater part of the city's milk supply. The growth of Detroit has been coincident with motor-vehicle and highway development and these agencies not only furnished the means of transporting the new supply but also took over a considerable portion of the business handled by the older agencies.

In the following table there is shown a comparison of the amount of milk transported by these different agencies during July, 1915, and during February, 1924.

Milk transported by different agencies, July, 1915, and February, 1924

	July, 1915		February, 1924	
	Pounds	*Per cent*	*Pounds*	*Per cent*
Steam railroads	5,363,415	41.8	775,710	2.1
Electric railroads	5,421,810	42.3	3,265,290	9.1
Motor trucks and wagons	2,038,130	15.9	31,743,690	88.8
Total	12,823,355	100.0	35,784,690	100.0

In 1924 when wagon shipments had ceased, 88.8 per cent of Detroit's milk supply was shipped by motor truck whereas only 15.9 per cent came in over the highways in 1915 by motor truck and by wagon. Steam and electric railroad shipments shrank from 84 per cent in 1915 to 11 per cent in 1924.

MOVEMENT OF MILK TO CITY LARGELY FROM RECEIVING STATIONS WITH LARGE TRUCKS PREDOMINATING AND LONG HAULS FREQUENT

With the constant increase in the mileage of improved roads the effective trucking radius has been correspondingly increased so that at present there are five trucks operating in the 70–79 mile zone. The numbers of motor trucks engaged in the transportation of milk according to mileage zones are shown in the table below.

Number of trucks in the various mileage zones, Detroit area

Mileage zone	Number of trucks	Per cent	Mileage zone	Number of trucks	Per cent
0–9			50–59	15	12.3
10–19	12	9.9	60–69	10	8.2
20–29	32	26.2	70–79	5	4.1
30–39	26	21.3			
40–49	22	18.0		122	100.0

Almost 65 per cent of the motor trucks bringing milk into the city operate over routes 30 miles and over in length. The large number of trucks in the higher mileage zones is a significant fact. Approximately 25 per cent of the trucks go beyond the 50-mile radius for their loads.

The 122 motor trucks engaged in this milk traffic are classified as to capacity in the following table.

Trucks classified by capacity, Detroit area

Capacity in tons	Number of trucks	Per cent	Capacity in tons	Number of trucks	Per cent
1	2	1.6	4	4	3.3
1½	4	3.3	5	37	30.5
2	12	9.6	7½	1	.8
2½	14	11.5			
3	27	22.2		122	100.0
3½	21	17.2			

Trucks of large capacity predominate in number and 65 out of the 122 are tank trucks. Approximately 75 per cent of the trucks have a capacity of 3 tons and over; 30.5 per cent are in the 5-ton class. The 7½-ton truck is used in hauling crates of bottled milk from a farm 20 miles outside of the city over a hard-surfaced road. The run is made in a little over two hours.

Only 12 of these trucks are engaged in pick-up business, hauling directly from the farm to the city dealer or dairy plant. The others are all operated between country receiving stations and city dairies.

The 12 trucks picking up milk along the road do not go beyond the 30-mile limit and are all of the smaller capacities.

In the Detroit milk shed there are 120 receiving stations where the milk is collected from the producers, put through a cooling process in the summer months, and shipped to the city dairies either by tank trucks or by large can trucks. There are 110 trucks engaged in this kind of traffic, 65 of which are tank trucks. It is generally stated that there are more tank trucks engaged in the transportation of milk in the Detroit area than in any other section of the United States. The establishment of such a large number of receiving stations in this section and the transportation of cooled milk in cans and tank trucks was largely caused by the ruling of the city health department requiring the milk to arrive at a temperature not above 60° F. The enforcement of this rule hastened the establishment of receiving stations and also the bulk method of transportation.

The majority of these receiving stations are owned and operated by the city dairy companies. A few are owned by farmers' cooperative associations and by individuals. The typical receiving station consists of a small frame building equipped with a small boiler, can-washing machine, and cooler. The larger stations have additional equipment for separating, churns, cheese vats, condensing pans, and ice machines. The smaller stations use natural ice for cooling. The milk is hauled to the stations in small trucks and wagons by the farmers and by local individuals who may engage in that business. The price paid for this hauling usually averages from 15 cents to 20 cents per 100 pounds depending on the roads and distances.

TANK TRUCKS USED FOR LARGE VOLUME MOVEMENT PARTICULARLY FROM DISTANT STATIONS

In cases where shipment to the city is made by tank truck the truck is run alongside the receiving station where it is filled either by gravity or by a small compressed air pump. The tanks are filled to capacity to prevent churning en route. Where receiving stations are located near to one another the tank trailer has been found very useful. The tank truck and the tank trailer proceed to the first station where the trailer is left to be filled while the tank truck goes on to the station beyond to get its load. On the return trip the tank trailer is picked up by the tank truck and both are taken to the city dairy.

Of the 65 tank trucks 35 are owned and operated by one concern which has a contract for this kind of hauling with one of the largest dairies in Detroit. The average daily hauling of these 35 tank trucks and the number of receiving stations they serve, and the various mileage zones are shown in the following statement.

Average amount of milk hauled daily with 35 tank trucks, mileage zones and number of receiving stations, Detroit area

Mileage zones	Number of stations	Average amount of milk hauled daily [1]	Average amount per station	Mileage zones	Number of stations	Average amount of milk hauled daily [1]	Average amount per station
		Pounds	*Pounds*			*Pounds*	*Pounds*
10–19	5	17,433	3,487	50–59	5	113,180	22,636
20–29	6	38,861	6,477	60–69	1	18,934	18,934
30–39	7	84,591	12,084	70–79	2	41,480	20,740
40–49	4	33,158	8,289				

[1] In some areas the gallon is the standard unit, while in others the pound or hundredweight is used. For purposes of comparison it may be remembered that 100 pounds of milk represents 11.7 gallons.

The average amount of milk hauled from these receiving stations daily ranges from 3,487 pounds for the stations in the 10–19 mile zone to 22,636 pounds per station in the 50–59 mile zone. It is observed that the daily amount moved from stations increases with the distance to the stations, with one exception.

Where trucks take over business formerly handled by railroads, the railroad rate is adopted as the trucking charge. The following are samples of motor-truck rates in effect for various distances:

Typical motor truck rates, Detroit area

Station	Distance in miles	Rate in cents per 100 pounds	Station	Distance in miles	Rate in cents per 100 pounds
Washington	30.0	27	Peters	38.0	30
Memphis	53.5	50	New Boston	27.3	45
Macomb	30.0	27	Waltz	25.5	45
Utica	21.0	24	Scofield	34.0	45

The hauling from the last three stations is performed by one concern, which charges a flat rate of 45 cents. In general, the rates are graded in such a manner as to approximate a charge of 1 cent per 100 pounds per mile,

Tank trucks are used principally for transporting milk from receiving stations to the city plant. A large fleet of such trucks is used in the Detroit area

though there are obvious exceptions. The survey did not show that any differentials were made as between tank trucks and other types of trucks used in the transportation of milk.

Practically every milk truck in the Detroit area employs the services of a driver only, which is to be expected where the entire load is secured at one point. Few trucks entering the city pick up milk along the road, and in no case is cooperative hauling done. There has also been no development of the return-load feature, with the exception of small shipments of ice cream taken back from the city dairies to points in the country. The use of tank trucks naturally eliminates the opportunity of developing city-to-country shipments of merchandise and other kinds of freight.

CITY OF MILWAUKEE LOCATED IN HEAVY MILK PRODUCING SECTION

The greater part of the milk supply of Milwaukee is received from the territory within 30 miles of the city. This takes in the whole of Milwaukee County, the northern part of Racine County, the eastern part of Waukesha County, and the southern parts of Washington and Ozaukee Counties. Lake Michigan on the east prevents any milk coming from that direction.

This half-circular territory is probably the heaviest milk-producing section in close proximity to any large city in the United States. Along the outskirts of the city's milk shed are found a string of condenseries and a number of receiving plants operated by Chicago milk distributors.

As late as 1916 half of the milk consumed came to the city by wagon and the other half was shipped by railroad. At the present time, as is shown by the table below, almost 90 per cent of the milk is transported into the city by motor truck and the wagon has been wholly displaced.

Number and per cent of cans transported daily by rail and motor truck, Milwaukee area

	Number of cans	Per cent
Motor truck	7,352	87.5
Electric railroad	1,055	12.0
Steam railroad	50	.5
	8,457	100.0

A type of body especially built for milk hauling. Note side opening, tight sides for protection against freezing, and canvas cover. Cans must be of uniform size for maximum load

The containers are all 8-gallon cans. These were the average daily receipts in the month of February, 1924, at a time when several of the outlying routes were closed to motor traffic so that the rail lines were at this time carrying slightly more than their normal share. It is noteworthy that the steam railroads which penetrate and serve this whole area carried only 50 cans of milk a day. The electric railroad has been able to hold its milk shipments largely because it has worked out a coordination of its rail service with motor trucks which act as feeders. It has made an arrangement with an operator of two motor trucks which collect on an average of 300 cans each and deliver at the milk platforms on the interurban line where they are picked up by a special train and hauled into the city.

That Milwaukee's milk trucks operate in a territory not far distant from the city is shown in the following table:

Number and per cent of trucks operating in various mileage zones, Milwaukee area

Mileage zone	Number of trucks	Per cent
0–9	5	4.0
10–19	71	56.8
20–29	45	36.0
30–39	4	3.2
Total	125	100.0

There are 125 motor trucks engaged in hauling milk directly to the city dairy plants; 76 trucks, or 60.8 per cent operate within the 0–19 mile zone; 45 trucks, or 36 per cent, operate in the 20–29 mile zone; only 4 trucks, or 3.2 per cent, operated over routes between 30 and 39 miles in length.

In the following table the motor trucks are divided both as to capacity and to mileage zones.

Number of trucks classified by capacity in the various mileage zones, Milwaukee area

Mileage zone	1-ton	1½ ton	2-ton	2½ ton	3-ton	3½ ton	5-ton	Total
0–9	4	1						5
10–19	8	10	21	20	3	6	3	71
20–29	11	8	5	10	9		2	45
30–39	2	1					1	4
Total	25	20	26	30	12	6	6	125
Per cent	20.0	16.0	20.8	24.0	9.6	4.8	4.8	100.0

Out of a total of 125 trucks only 6 or 4.8 per cent are of the 5-ton class. Over 56 per cent of the trucks are in the 1 to 2 ton class. On account of the small area over which they operate it can not be said that the small-capacity trucks are found only in the low-mileage zones and the larger trucks in the higher-mileage zones. The tabulation shows that one of the 5-ton trucks operates in the 30–39 mile zone and there are two 1-ton trucks operating over the same length of route.

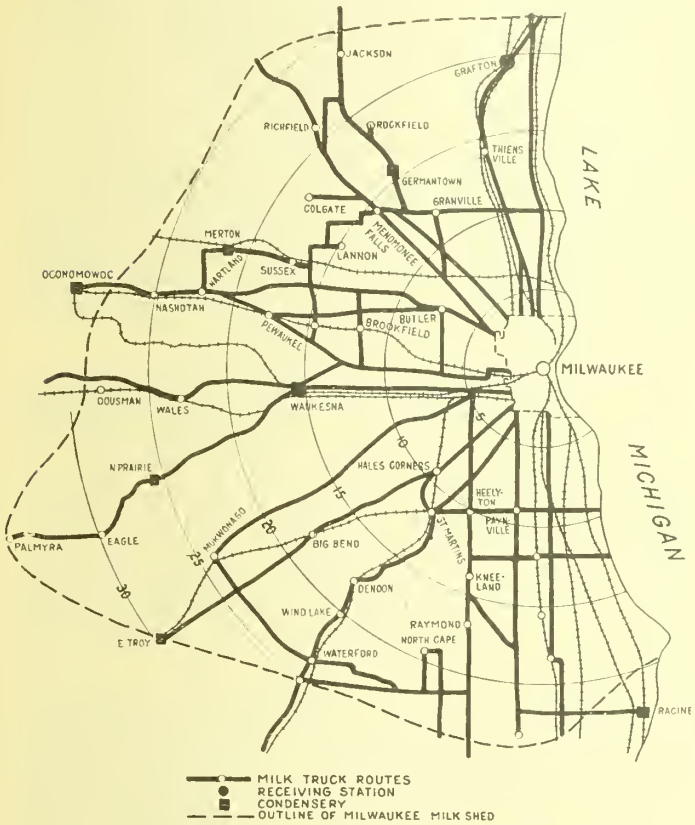
Most of the trucks are equipped with bodies of light board construction to prevent the freezing of milk. Where the ordinary stake bodies are used there is much trouble on this account in winter. Bodies of sufficient height to allow plenty of room for double-decking a load of standard 8-gallon cans are used. All the cans being of the same size, the hauler is enabled to load his truck to maximum capacity. The bodies have side gates so that the trucks can readily load from milk platforms on both sides of the road.

RECEIVING STATIONS NOT NECESSARY IN MILWAUKEE AREA

The large volume of milk which can be gathered in the territory close to the city makes the establishment of receiving stations unnecessary. The rates of the small trucks delivering milk to receiving stations are practically as high as those charged for shipment to the city milk plants. An experiment was made with the operation of a tank truck from a receiving station 12 miles from the city but the operation was finally discontinued when the station haulers demanded nearly as much per 100 pounds as direct city haulers were getting f. o. b. city plants. With the excellent system of hard-surfaced highways possessed by the Milwaukee milk shed it is doubtful whether receiving stations can be successfully operated within the present confines of the milk-producing territory.

The milk trucks in this territory are owned mainly by individuals who live in the country at or near the point of origin of their respective routes and who farm a few acres after the daily milk hauling is completed. These farmer-truck operators, who compose about 90 per cent of the total number, usually own but one truck, sometimes two. There are no large fleet organizations such as operate in the Detroit milk shed. The trucks are usually operated by the owner and there is seldom a helper employed. The owner and operator of the truck acts in the capacity of the farmer's agent rather than the dealer's. The hauling contract is made between the hauler and the producer and not between

the truck owner and the dealer. The dealer, however, deducts the hauling charges from each producer's monthly milk check and remits these to the truck operator thus saving the hauler the trouble of collecting a number of small accounts.



Map of Milwaukee milk shed showing routes and mileage zones

Twelve of the trucks engaged in milk traffic are owned and operated by one of the largest dairy companies. This company employs several of its smaller trucks as feeders to the larger trucks operating over longer routes.

Milk hauling is an everyday business. The milk must move in the winter as well as in the summer, on stormy days as well as fair. Consequently steps have been taken to assure the regularity of truck movements despite heavy snowfalls. Milwaukee is fortunate in having practically all of its incoming main country highways constructed with a hard surface, and the efficient snow removal by the counties from which the milk comes has permitted trucks to reach the city in all kinds of weather. Snow removal in Wisconsin is left to the county authorities. Drift fences have been a big aid at several points and snow plows are placed in operation as soon as the snow commences to fall. These are kept operating without stopping until the snow ceases. During extremely heavy snow conditions and when the frost is coming out of the ground the milk is brought by the producers over the dirt roads to the main hard-surfaced roads where the regular trucks pick it up from platforms.

The return-load traffic has not been developed to any extent in the Milwaukee area. Most of those operating the milk trucks have a good paying load of milk and do not seem to care to develop this additional business. Because of the relatively short routes many of the trucks make two trips with milk and others haul produce to market in the afternoon after they have returned the empty milk cans from the morning trip.

HIGH RATES CHARGED BY TRUCKS TRAVERSING BAD ROADS

The rates charged for hauling vary for trucks covering the same length of route. In order to give a complete picture of the rate schedules there are shown in the following tabulation the exact rates charged by each of the trucks and the length of the routes and also the average rates for each of these instances.

Miles	Rates in cents per 100 pounds charged by each truck	Average rate, cents per 100 pounds
7	15	15
8	16, 20	19.6
10	15, 15	15
11	18	18
12	15, 23	19
13	22, 20, 16, 23, 23, 25	21.5
14	21, 23, 15, 20, 20, 20, 23, 23, 15, 24, 23, 23, 21, 20	21
15	23, 23, 22, 20, 23, 23	22.3
16	23, 23, 23, 23, 23, 22, 23, 21, 23	22.6
17	23, 23	23
18	20, 25, 24, 25, 22, 23, 25	24
19	24	24
20	27, 23, 20, 25, 23, 22	23.3
21	30	30
22	23, 25, 30, 23, 25, 30, 23	25.5
23	23, 23, 25	23.6
24	25, 23, 21, 23	23
25	30, 30	30
26	28	28
27	28, 26	27
28	30, 25, 30	28.3
30	30, 30	30
34	35	35
39	40	40

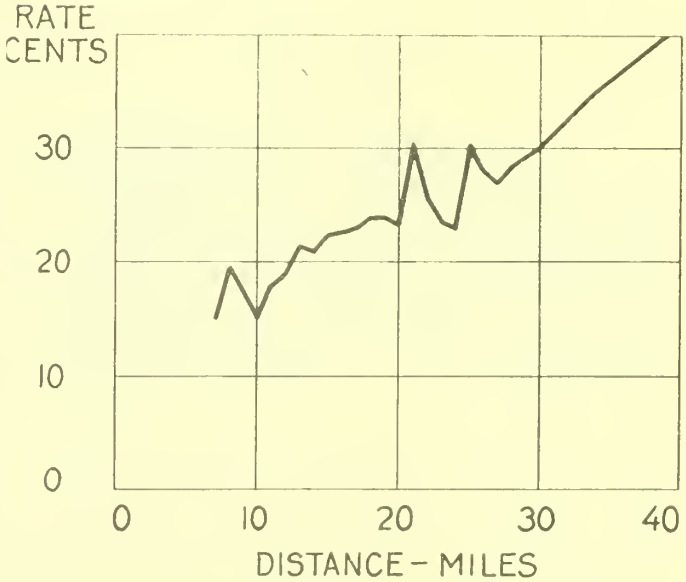


Diagram showing average rates per 100 pounds for hauling milk by truck for varying distances in Milwaukee area

The rates average from 15 cents per 100 pounds for a short haul of 7 miles to 40 cents for a 39-mile haul. For distances of 27 miles and over the rate approximates 1 cent per 100 pounds per mile. For the shorter distances it is generally higher. Up to 10 miles the rate is practically 2 cents per 100 pounds per mile and for distances above that it is in the neighborhood of 1½ cents. There are, however, glaring inconsistencies in some of the rates charged for specific distances. It is usually found that where there are higher rates than others for similar distances the trucks charging the higher rates collect their loads in sections which are

not provided with hard-surfaced roads and then have some distance to go before they strike the main improved highway leading into the city. As an illustration, there are two trucks operating a distance of 12 miles; one charges 15 cents, starts at a point on a hard-surfaced road and continues over it until it reaches the city; the other truck, which charges 23 cents going the same distance, proceeds for about 4 miles before it reaches a concrete road. Similarly the two trucks which charge a rate of 15 cents for a distance of 14 miles operate over a hard-surfaced road the entire route; the other trucks operating over a like distance charge higher rates and in most cases follow routes which are not confined solely to the higher types of roads. There are undoubtedly other reasons which may account for these differentials in charges for the same distances, but the condition of the highway appears to be the important factor and accounts for most of the variations.

CHEAP TRANSPORTATION BENEFITS BOTH PRODUCER AND CONSUMER

In the following table is shown a comparison of net prices received at the farm in a number of milk markets of the country in 1923:

Average milk prices, 1923

City	Average net price per 100 pounds at the farm	City	Average net price per 100 pounds at the farm
Philadelphia	$2.46	Columbus	$2.08
Cincinnati	2.02	Pittsburgh	2.34
Detroit	2.232	Cleveland	2.50
St. Paul and Minneapolis	2.21		
Chicago (estimated)	1.60	Average	2.152
Louisville	1.87	Milwaukee	2.39

The average retail selling price in these cities, excluding Milwaukee, was 12.9 cents per quart; in Milwaukee the average retail selling price for 1923 was 10.41 cents per quart. This indicates that the producers in the Milwaukee shed received 11.5 per cent more than the producers supplying these other markets and the consumers paid only 81 per cent of what the consumers paid in these other cities. The higher price received by the producers is to a large extent accounted for by the production of such a large quantity of milk in an area close to the market, thereby reducing the amount which has to be paid out for transportation. The high-type roads in this area are factors in holding haulage charges down to a low level. Extensive use of motor trucks in the transportation of milk has eliminated the necessity of city hauling from railroad terminals to dairy plants. This was an expense which the distributors had to bear and its elimination is one of the factors which enables them to sell at a lower price than that charged in other cities.

PRACTICALLY ALL MILK NOW SHIPPED BY MOTOR TRUCK IN ST. PAUL AND MINNEAPOLIS

From the standpoint of the transportation and marketing of milk St. Paul and Minneapolis, commonly called the Twin Cities, are generally considered as one market. The producers who supply these cities are organized and are known as the Twin City Milk Producers' Association, composed of over 6,000 members. The association controls about 80 per cent of all the milk produced and sold within the milk shed of these two cities which lies within a radius of 40 miles. On account of large quantities of surplus milk at various seasons of the year the association has found it necessary to operate 15 receiving stations which are equipped to manufacture one or several milk products, such as butter, cheese, casein, and condensed milk. During the late summer and the fall months, when the production of milk is lower than at any other time of the year, these stations may be shipping their entire supply into the Twin Cities to meet the demands of the distributors. During March, 1924, the association handled 23,186,496 pounds of milk. Only half of this was sold as fluid milk to the city distributors, the rest being converted into milk products at these country stations. Consequently only 50 per cent of the milk bought by the association was transported to the Twin Cities, thereby saving that part of the hauling cost.

Of the total amount of milk handled by the association only about 1 per cent is shipped by railroad; the remainder is hauled direct by motor truck to the plants of the city distributors or to the nearest receiving stations of the association. There are now only four rail shipping points, located from 30 to 47 miles distant, and these are not on any of the State highways. There are still a small number of farmers who haul their milk directly by wagon or truck to city dealers or in some cases they do their own distributing from door to door. The milk brought in by this method is only a small portion of the consumption of these two large cities.

At the present there are 122 motor trucks engaged in hauling milk directly to the city dairies or to the two central plants maintained in the Twin Cities by the association. These plants are operated by the association as pooling plants to which various city dealers come for the milk which they distribute. The number and per cent of trucks in the various mileage zones is shown in the following table.

Number and per cent of trucks in various mileage zones, Twin Cities area

Mileage zones	Number of trucks	Per cent
0–9	11	9.0
10–19	45	36.9
20–29	54	44.2
30–39	12	9.9
Total	122	100.0

Practically 90 per cent of the trucks are operated within the 0–30-mile zone and the remaining 10 per cent are in the 30–39-mile zone. In addition to these trucks there are nearly 60 station haulers who gather milk within a radius of 10 to 15 miles from the various receiving stations. The milk produced within 25 miles of the Twin Cities is usually taken directly to the city dairy plants or to the association's two central plants. Milk produced in the territory beyond is diverted to the country receiving stations where it is cooled and reshipped to the Twin Cities or manufactured into milk products. There are only two tank trucks operating in this territory. These are used by one of the distributors in transporting his supply of milk from country receiving stations to his city plant.

The classification of the motor trucks engaged in the milk traffic by capacity is set forth in the following tabulation.

Number and per cent of trucks by capacity, Twin Cities area

Capacity in tons	Number of trucks	Per cent	Capacity in tons	Number of trucks	Per cent
1	76	62.3	3	5	4.1
1½	12	9.9	3½	4	3.2
2	20	16.4			
2½	5	4.1		122	100.0

The large number of trucks of 1-ton capacity is striking; 62.3 per cent of the total number are in this group. Almost 90 per cent of the trucks are of 1 to 2 ton capacity. Practically all of the trucks are fully equipped with pneumatic tires.

TRUCK RATES HIGHER THAN FOR OTHER AREAS

The average rates charged for hauling milk by motor truck are set forth in the table following.

Transportation rates charged by trucks, Twin Cities area

Distance in miles	Average rate in cents per 100 pounds	Distance in miles	Average rate in cents per 100 pounds
0–9	22.5	20–29	33.0
10–19	28.0	30–39	37.5

These rates are higher than those in several other milk sheds. Higher rates are partly due to road conditions but can not be entirely ascribed to this cause. The general use of pneumatic tires results in higher operating expenses. It is claimed by those engaged in this business that they are necessary in order to operate successfully over the snow-covered roads during the winter months. Many haulers quote a winter and a summer rate. Others maintain a uniform rate which they feel is high enough on the average to cover their added costs during the winter months.

The milk trucks in this territory are owned and operated by individuals living in the country where their routes originate, 81 operate 1 truck each, 16 operate 2 trucks, and 3 operate 3 trucks each. This indicates that there is no developed fleet operation. Several milk trucks are operated by country storekeepers who use their trucks in bringing back return loads of merchandise and the return load movement is confined to this class.

Under a statute recently passed these milk trucks operating over regular routes and on fixed schedules are classed as common carrier vehicles and have to pay as a tax and license fee 10 per cent of the value of the truck. The "base price for taxation" is obtained from a statement filed by truck manufacturers, "less 10 per cent for the second year, less 20 per cent for the third," and so on until the eighth year when it is 70 per cent for that year and for each subsequent year. The old rate was based on an assessment of 2¾ per cent of the value.

MOTOR TRUCK HAS GROWN TO PRINCIPAL AGENCY FOR MILK TRANSPORTATION IN INDIANAPOLIS

Although Indianapolis is located in the heart of a rich agricultural district, the production of market milk in the counties tributary to the city has always been limited in quantity until recently. City distributors in order to meet the demands of the market were forced to reach out distances of 40 to 50 miles which meant the establishment of receiving stations. The milk, after it was cooled, was shipped in cans from these stations to the city by steam and electric railroads.

Highway transportation has caused a great change in the last five years, developing a producing area which is much closer to the city. Receiving stations are gradually being closed. One large dairy company has closed 8 out of 12 stations and expects to close 1 or 2 more this year. The producing area or milk shed has in this case grown smaller rather than larger. For both the producer and the city distributor the use of the motor truck in hauling milk has greatly simplified operations.

The average amount of milk transported into Indianapolis daily during the month of March, 1924, is shown in the following statement:

Amount of milk transported into Indianapolis, daily average, March, 1924

	Amount of milk in pounds	Per cent
Motor truck	224,458	94
Electric railroad	14,450	6
Total	238,908	100

A receiving station equipped to convert surplus milk into other products

Five years ago the electric railroads carried 50 per cent of the milk supply, wagons 30 per cent, steam railroads 10 per cent, and motor trucks 10 per cent. At the present time no milk is received by steam railroad and the traction lines bring in only 6 per cent of the total supply. The traction lines lost this kind of traffic largely because the motor truck furnished a complete service from the point of production in the country to the city dairy plants.

The number of trucks engaged in this service and the mileage zones are shown in the following statement:

Number of motor trucks operating in the various mileage zones

Mileage zones	Number of trucks	Per cent	Mileage zones	Number of trucks	Per cent
0–9	4	4.7	30–39	17	19.8
10–19	26	30.2	40–49	10	11.5
20–29	29	33.8	Total	86	100.0

This table shows that over a third of the trucks operate in the 0–19 mile zone and another third in the 20–29 mile zone. Two-thirds of the number of trucks engaged in this traffic operate in the area within 30 miles of the city.

In the following table is shown the number of trucks by capacities operating in the various mileage zones:

Number of trucks by capacities in various mileage zones

Mileage zones	1-ton	1½-ton	2-ton	2½-ton	3-ton	Total
0–9	4					4
10–19	11	10	4	1		26
20–29	17	8	4			29
30–39	14	1	2			17
40–49	7	2			1	10
Total	53	21	10	1	1	86
Per cent	61.6	24.4	11.6	1.2	1.2	100.0

The large number of small trucks operating in this territory is significant; 86 per cent are 1 to 1½-ton capacity; 97 per cent are 2-ton or less. All the trucks are of the open-body type except one 3-ton truck which is of the closed-van type and operates from a receiving station over a 48-mile route.

In the following table are set forth the average rates charged by the trucks and by one of the electric lines.

Rates charged per 100 pounds by trucks and by electric railroads, Indianapolis area

Distance	Truck rates per 100 pounds	Electric railroad rates per 100 pounds	Distance	Truck rates per 100 pounds	Electric railroad rates per 100 pounds
Miles	*Cents*	*Cents*	*Miles*	*Cents*	*Cents*
5–10	30	19.84	30–35	45	25.73
10–15	33	21.32	35–40	48	26.46
15–20	36	22.78	40–45	50	27.20
20–25	39	23.52	45–50	50	28.67
25–30	42	24.26			

The truck rates for the shorter distances average considerably higher than the rates charged in other territories for short hauls. For the longer distances the rates are slightly more than a cent per 100 pounds per mile which seems to be a normal scale in other cities. Although the truck rates are maintained on a higher level than the traction-line rates still shipments by motor truck have been increasing.

Return loads are seldom carried by the milk trucks in this territory. The trucks are practically all owned by individuals living in the country who may be farming on a small scale. The license fees paid are the same as are paid by other trucks not engaged in regular service.

CHARLES M. UPHAM APPOINTED DIRECTOR OF ADVISORY BOARD ON HIGHWAY RESEARCH

CHAS. M. UPHAM, State Highway Engineer of North Carolina, has been recently appointed director of the Advisory Board on Highway Research of the National Research Council, to succeed Dr. W. K. Hatt who resigned in order to resume his work at Purdue University.

Charles M. Upham

The present board intends to continue the excellent plans already effected and to extend its activities so that the results of highway research may be practically applied by the States and counties carrying on programs of highway construction and maintenance and by others interested in highways. The organization has been extended so that each State highway department may have a representative on the board who will serve as a point of contact between it and the State. It is also planned to have similar representation from universities engaged in highway research.

Mr. Upham, the new director, has had extensive experience in highway work. He received his early training with the Massachusetts highway department and later became chief engineer of the Coleman du Pont Road in Delaware. Following this he was for four years chief engineer of the Delaware State Highway Department, when he was called to take charge of the extensive highway construction program in North Carolina. Mr. Upham holds a B. S. degree from Tufts College and an honorary C. E. degree from the University of North Carolina. He is an associate member of the American Society of Civil Engineers and holds active membership in many other technical societies. For the past two years he has been business director of the American Road Builders' Association, and has been reelected several times to his present position as secretary of the American Association of State Highway Officials.

It is expected that Mr. Upham's wide acquaintance and broad experience in the highway field will make his connection as director of the Advisory Board on Highway Research especially valuable to those agencies which can utilize the information being made available by this organization.

The offices of the board are now located in the new and imposing building of the National Research Council at B and Twenty-first Streets, Washington, D. C.

FRICTION TESTS OF CONCRETE ON VARIOUS SUB-BASES

BY A. T. GOLDBECK, CHIEF, DIVISION OF TESTS, UNITED STATES BUREAU OF PUBLIC ROADS

THE rational design of any structure, whether it be a bridge or a pavement, necessitates a knowledge of the stresses to which it will be subjected. In a framed structure, stress analysis is comparatively simple, and may be made with practical surety. When one attempts, however, to analyze a concrete road slab, a bewildering maze of uncertain forces is found. The uncertainty lies not so much in not knowing what forces exist as in an almost total lack of knowledge regarding their magnitude. We know that a concrete road slab is acted upon by forces which tend to slide it along its supporting base, which bend it, warp it, subject it to direct compression or tension, or to a combination of direct stresses and bending, forces of shear which tend to abrade its surface, and disrupting forces which weaken it locally. The series of tests discussed in this article was made to throw light on at least one of the forces acting on concrete roads, namely, the force of friction at the base.

It has been shown by several investigators that concrete elongates and shortens due to two principal phenomena; change in temperature and change in moisture content. A coefficient of 0.0000055 per degree Fahrenheit seems to accurately express the effect of temperature change. Tests [1] made on a number of large specimens of concrete demonstrate that moisture also plays an important part in influencing the length of concrete structures. Measurements made on new concrete after hardening and repeated at a later date indicate that drying out may cause a shrinkage of as much as 0.06 per cent while specimens kept in a moist condition may expand 0.01 per cent. Both old and new concrete if subjected to alternately wet and dry conditions must inevitably expand and contract. Carefully conducted field measurements [2] made with a specially designed instrument corroborate laboratory measurements and show that concrete pavements are subjected to considerable motion over their subbase.

DETERMINATION OF FRICTIONAL RESISTANCE A STEP TOWARD RATIONAL DESIGN OF CONCRETE PAVEMENTS

Heavy masses of concrete can not be slid over the ground without considerable frictional resistance which will vary with the kind and conditions of subbases. In order to study the effect of the movement of pavements on the internal stresses developed, let a hypothetical case be considered in which the pavement after being laid is subjected to a rise in temperature and in addition, due to weather and subbase conditions, is not allowed to dry out. Such a pavement will necessarily expand and the ends of the slab will move away from the center. Friction will act to resist this movement and in this way becomes an external force producing almost direct compression in the concrete. Should the frictional resistance exceed the compressive strength of the concrete, a compression failure will result.

Assume another case in which the temperature decreases considerably after the laying of the pavement, and let the concrete dry out due to dry weather and a well-drained subbase. This concrete will shrink and the ends of the slab will approach its center. Friction will act against the movement and tension will be produced in the concrete. Concrete is weak in tensile resistance and not much shrinkage is required to crack the pavement if the subbase is very rough. From these considerations it will be recognized that as a step toward the rational design of concrete pavements the determination of frictional resistance at the base is necessary, and the following investigation was made with this in view.

DESCRIPTION OF SPECIMENS AND METHOD OF TESTS

The bases of concrete roads have been of varying character. Loam, clay, old macadam, gravel, sand, and many other kinds of material support the concrete, and some offer more frictional resistance than others. In planning the series of tests, therefore, a number of different bases were used. Shallow ditches 6 inches deep, 3 feet wide, and 7 feet long were first dug in clay soil at the Arlington Experimental Station of the Bureau of Public Roads. Filling material forming the subbases was then deposited, tamped solidly, and smoothed or otherwise treated in readiness for placing the concrete. Upon the subbases slabs were cast 2 feet wide, 6 feet long, and 6 inches thick of 1:1½:3 concrete, machine mixed, of medium-wet consistency. The various subbases prepared were as follows:

A. Clay, smooth top surface.
B. Clay with cobblestones partly rolled in surface.
C. Broken stone, three-fourths inch to dust, flat top surface.
D. Concrete base, top surface troweled smooth.
E. Loam, smooth top surface.
F. Sand, top surface smoothed and oiled with heavy flux oil.
G. Clay, surface scored to make it uneven.
H. Gravel, three-fourths inch to one-fourth inch, flat surface.
I. Broken stone, 3-inch
J. Concrete base, troweled surface, oiled with heavy flux oil.
K. Sand, surface smooth.
L. Clay, oiled with heavy flux oil.

The first set of tests was made when the specimens were one month old. A frame made of a piece of one-half-inch round steel bent to surround the specimen was fastened to the pulling chain, which in turn was linked to a spring dynamometer. Force was applied by means of a light steel rail used as a long lever. Two men at the end of the lever were able to apply a constantly increasing force with great steadiness. The movements of the slab were read by means of a Berry strain gauge, and in this way the movements corresponding to the force applied as indicated by the dynamometer were obtained.

The forces required to slide the specimens, together with their corresponding movements are shown in the diagram. Each specimen weighed approximately 870 pounds. The ground during these tests was damp but very firm. It will be noted that movement takes place almost as soon as the load is applied in all cases except

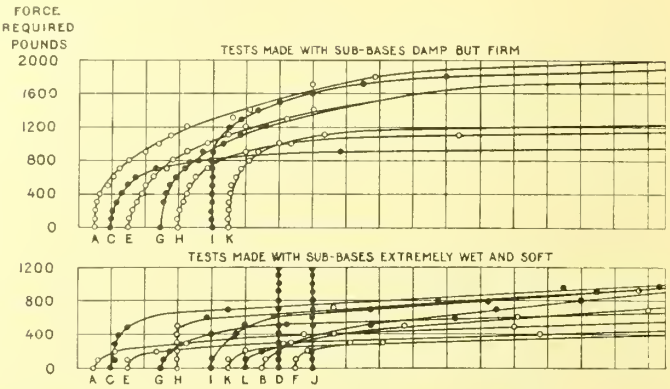
[1] The Expansion and Contraction of Concrete while Hardening, by A. T. Goldbeck, 1911 Proceedings of American Society for Testing Materials.
[2] See United States Department of Agriculture Bulletin No. 532, The Expansion and Contraction of Concrete and Concrete Roads.

that in which large broken stone was used in the base. Here there is great friction from the start until finally when a load of 1,000 pounds is reached slipping begins to take place. Great force was required to start the slab, after which no more was necessary than in some of the other bases. Apparently the coefficient of friction is not constant but varies with the displacement. When the slab on the loam base had been slid 0.035 inch, the load was gradually released, and the return movement noted on the strain gauge. The subbase seemed to behave in a somewhat elastic manner as the slab actually recovered considerable of its forward movement. This can be accounted for only by the partial springing back of the subbase toward its original position, carrying the slab along with it. The values of the frictional resistance offered at various displacements are given in Table 1.

FRICTIONAL RESISTANCE LOWERED BY WET SUBBASE

The same specimens and a few more that had been completed in the meantime were again tested after a period of about a month and a half. During this interval the ground was frozen solid almost all of the time, but the day before the tests a thaw set in, and combined with rain served to make the ground exceedingly muddy. The bases of most of the specimens were thoroughly saturated with water which stood on the surface in small puddles. The results obtained under such conditions are shown in the diagram, and it will be noted that the frictional resistance is considerably lower than it was in the previous tests. The water acted as a lubricant and permitted the slabs to slip easily. The values of the frictional resistance offered at various displacements are given in Table 2.

The results in the latter table, when compared with those given in the previous table, show very clearly that much depends upon the moisture condition of the subbase. A wet subbase permits the concrete to slide very much more easily than does a dry subbase. This apparently applies also to the specimens placed on broken stone and gravel bases, particularly when the movements are small.



Movement of concrete slabs on various subbases due to application of horizontal forces. Horizontal divisions represent movements of two one-thousandths of an inch. All slabs were 2 feet wide, 6 inches thick, and weighed 870 pounds

In order to try the effectiveness of a heavy oil coating for decreasing the friction at the subbase, several specimens were made having sand, clay, and concrete subbases treated with a heavy flux oil. The oil was applied while hot and was spread over the surfaces with a broom in a layer of sufficient thickness to completely cover the subbase. The concrete was deposited on these oiled bases immediately. So far as can be noticed, the oil is not effective in decreasing the friction in any of the bases, even when applied to the concrete base that had been carefully troweled smooth. It was impossible to slide the concrete specimen on the concrete base, even when a load of 2,500 pounds was applied. No attempt was made to apply a greater load than this, because 2,500 pounds was the limit of the dynamometer. When the maximum load had

(Continued on page 23)

TABLE 1.—*Frictional resistance of concrete on various subbases in damp but firm condition*

Kind of base	Movement	Force	Coefficient	Movement	Force	Coefficient	Movement	Force	Coefficient
	Inches	*Pounds*		*Inches*	*Pounds*		*Inches*	*Pounds*	
Level clay	0.001	480	0.55	0.01	1,130	1.3	0.05	1,800	2.07
Uneven clay	0.001	500	0.57	0.01	1,120	1.29	0.05	1,800	2.07
Loam	0.001	300	0.34	0.01	1,030	1.18	0.05	1,800	2.07
Level sand	0.001	600	0.69	0.01	1,080	1.24	0.05	1,200	1.38
½-inch gravel	0.001	450	0.52	0.01	960	1.10	0.05	1,100	1.26
¾-inch broken stone	0.001	380	0.44	0.01	800	0.92	0.05	950	1.09
3-inch broken stone	0.001	1,060	1.84	0.01	1,550	1.78	0.05	1,900	2.18

TABLE 2.—*Frictional resistance of concrete on various subbases in thoroughly saturated condition with water and surrounding ground exceedingly soft*

Kind of base	Movement	Force	Coefficient	Movement	Force	Coefficient	Movement	Force	Coefficient	Movement	Force	Coefficient
	Inches	*Pounds*		*Inches*	*Pounds*		*Inches*	*Pounds*		*Inches*	*Pounds*	
Level clay	0.001	120	0.14	0.01	300	0.35	0.05	500	0.58	1.5	950	1.09
Uneven clay	0.001	200	0.23	0.01	460	0.53	0.05	620	0.71	1.4	925	1.06
Loam	0.001	150	0.17	0.01	260	0.3	0.05	410	0.47	0.75	925	1.06
Level sand	0.001	140	0.16	0.01	280	0.32	0.05	400	0.46	0.75	875	1.00
½-inch gravel	0.001	510	0.58	0.01	640	0.73	0.05	950	1.01	0.5	1,050	1.2
¾-inch broken stone	0.001	400	0.46	0.01	660	0.76	0.05	940	1.08	2.0	1,160	1.33
3-inch broken stone	0.001	240	0.28	0.01	630	0.73	0.05	900	1.04	0.875	1,625	1.87
Oiled clay	0.001	150	0.17	0.01	410	0.47	0.05	850	0.98	1.25	1,425	1.64
Clay and cobble stones	0.001	140	0.16	0.01	410	0.47	0.05	710	0.82	1.75	1,260	1.45
Concrete base	0.001	2,500+	2.9+	0.00	2,500+	2.9 +	0.00	2,500+	2.9 +	0.00	2,500+	2.9 +
Sand, oiled	0.001	180	0.21	0.01	280	0.32	0.05	480	0.55	0.375	800	0.92
Concrete, oiled	0.000	2,500+	2.9+	0.00	2,500+	2.9 +	0.00	2,500+	2.9 +	0.00	2,500+	2.9 +

THE BULKING OF MOIST SANDS

EFFECT OF PHENOMENON ON STRENGTH AND YIELD OF CONCRETE

By Arthur A. Levison, Highway Engineer, United States Bureau of Public Roads

When dry sand is moistened by mixing with a moderate amount of water, the volume of the sand is increased out of all proportion to the amount of water added. Stated otherwise, dry sand weighs more per cubic foot than moist sand. While this bulking or swelling phenomenon was recognized and investigated more than 30 years ago, the direct effect of it on the measurement of fine aggregates for concrete has attracted attention only recently. The Iowa State Highway Commission, for example, experimented in 1923 with the weighing of aggregates for concrete road construction and, as a result of the experiment, has adopted the practice of weighing as standard procedure. One of the advantages claimed for the weighing method is that the effect of moisture on the volume of fine aggregates can be disregarded, the only correction necessary being that for the weight of contained water. The inundation of sands has also been advocated as a means of measuring accurately under almost constant conditions the amount of sand and water for each batch of concrete.

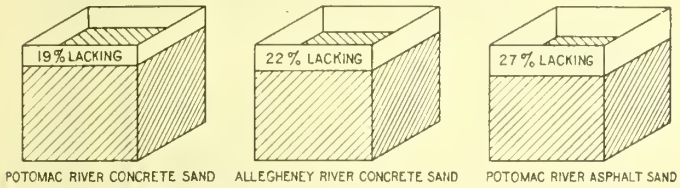


FIG. 1.—Volume of sand obtained by drying out 1 cubic foot of moist sand

Experiments made recently by the United States Bureau of Public Roads show that the extent to which a given sand will bulk is dependent principally upon its nature and gradation, the manipulation, and the amount of moisture present. In general, fine sands bulk more than coarse sands. Figure 1 shows the shrinkage of three widely different sands, the volume of which, measured in a condition of maximum bulking, was 1 cubic foot. It is apparent that volumetric measurements of concrete sands under certain moisture conditions will give approximately 20 per cent less of actual sand than the same volumes of the sand when measured in a dry condition.

BULKING AMOUNTING TO MORE THAN 25 PER CENT POSSIBLE WITH ORDINARY MOIST SAND

The relation between increase in volume and percentage of moisture is shown by the curves in Figure 2, from which it is evident that the rate of change of bulking is most rapid for moisture contents up to 3 or 4 per cent. Sands used for concrete construction are almost never dry and 3 per cent of moisture constitutes a barely damp condition. Even with 6 per cent of moisture there is no visible evidence of free water. On a 20-mile concrete paving job in Iowa where the aggregates were weighed, moisture determinations made several times each day showed that the moisture content of the sand ranged from 3 to 6 per cent with by far the majority of the determinations lying between 4 and 5 per cent. If the conditions on the Iowa job are at all representative, it will be apparent that for sands C and M, laboratory bulking factors of 22 and 28 per cent, respectively, will be sufficiently accurate

PERCENTAGES RETAINED ON STANDARD SIEVES	SAND "C" POTOMAC RIVER CONCRETE SAND	SAND "M" ALLEGHENY RIVER CONCRETE SAND	SAND "F" POTOMAC RIVER ASPHALT SAND
¼ INCH	1	0	0
NO 10	48	13	1
NO. 20	70	24	7
NO 30	79	48	19
NO.40	83	68	31
NO. 50	88	89	49
NO 80	93	97	70
NO.100	95	98	82
NO.200	97	99	92
PASSING NO 200	3	08	8

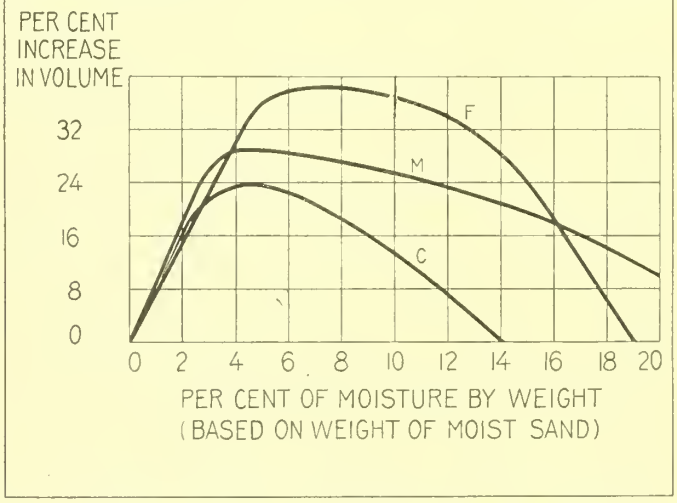


FIG. 2.—Curves showing the bulking of three different sands due to various percentages of moisture

for moisture contents between 3 and 6 per cent. Another interesting fact evidenced by the bulking curves is that it was possible with sands C and F to incorporate sufficient water to bring the sands back to an equivalent of the dry bulk. In the case of sand M this could not be done by the methods adopted.

The arrangement of apparatus for the laboratory test is shown in Figure 3. A standard slump cone was supported in an inverted position so that its bottom was 12 inches above the top of a one-half cubic foot metal container. A weighing scale, shovels, scoops, and an oven or some convenient means of drying small samples of wet sand completed the necessary equipment. The first step in a test was to determine the weight of one-half cubic foot of the dry sand when poured through the slump cone into the container without tamping. About 2 cubic feet of dry sand was spread on a con-

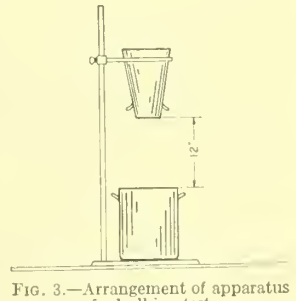


FIG. 3.—Arrangement of apparatus for bulking test

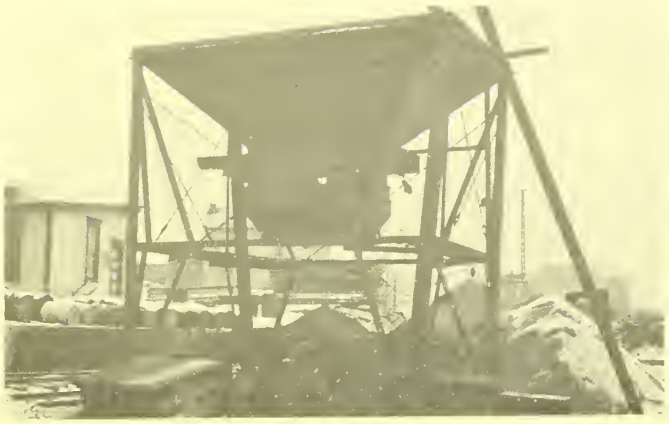


FIG. 4.—Steel bins and batchers used to determine field bulking of sand

crete floor and, beginning with a small amount of water, successive increments were added so as to raise the moisture content of the sand by steps of approximately 1 per cent. After each increment of water, the sand was thoroughly mixed and agitated with shovels until the moisture was uniformly distributed throughout the pile. When each increment of moisture has been thoroughly incorporated, a representative sample of moist sand was taken to be weighed and dried for determination of the percentage of moisture based on the weight of the moist sand, and two or more determinations of the weight per one-half cubic foot were made. This latter operation was performed after dropping the moist sand through the cone until the container overflowed, the excess being struck off with a straightedge. The entire process was repeated until the sand became saturated and water began to drain off the pile. The computation for bulking was made on a weight basis by deducting the weight of water from the weight of one-half cubic foot of moist sand and finding the difference between this weight and that of one-half cubic foot of the dry sand. Dividing this difference by the weight of dry sand in one-half cubic foot of moist sand, the percentage of bulking was obtained.

LABORATORY TESTS CLOSELY CHECKED BY FIELD EXPERIMENTS

The effect of moisture on the volume of sand having been determined in the laboratory, the question arose as to what amount of bulking actually took place in the field on a construction job. For the purpose of obtaining this information, experiments were conducted with a modern concrete proportioning plant consisting of steel bins and adjustable measuring batchers with the constant strike-off feature.[1]

The measuring batcher was first calibrated for capacity at various settings of the adjustable feature with dry sand, the capacities being based on the weights of dry sand contained in the batcher. Normal stock pile sand was then loaded into the bin and, with the batcher set for a known cubical capacity as determined by the calibration test, it was loaded with the moist sand and the contents were then dumped and weighed. Deducting the weight of the water, the resultant amount of dry sand in a known volume was found. From this the bulking of the sand under plant conditions was computed. Incidentally, it was also found possible to compute the required setting of the adjustable batcher which would give a capacity of moist sand equivalent in weight to the original capacity in dry sand. The above-described work was done with Allegheny River sand, designated as M in Figure 2, and the bulking under field conditions was found to be 3 per cent less than that in the laboratory. The normal condition of the stock pile sand was 5.5 per cent moisture content. Owing to the fact that the experimental work was done during February and March and that the plant was out of doors it was not found practical to reduce the moisture content below this percentage. Field experiments were made with moisture contents greater than 5.5 per cent up to a maximum of 9 per cent and for this range the amount of bulking did not vary any more than is shown by the bulking curves in Figure 2. A photograph of the plant used in the field tests is shown in Figure 4.

[1] This work was made possible through the courtesy and cooperation of the Blaw-Knox Co. of Pittsburgh, Pa.

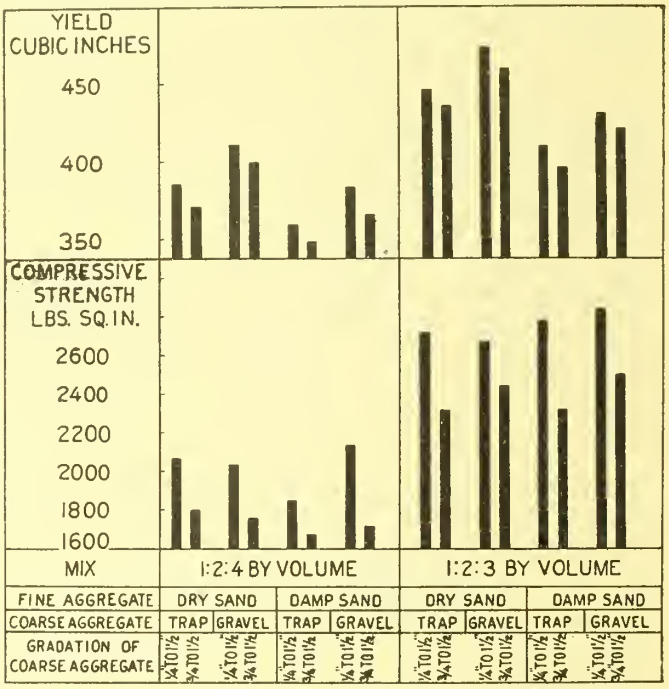


FIG. 5.—Results of tests on concrete cylinders, showing effect of bulking of sand due to moisture and gradation of coarse aggregates on yield and strength of concrete

THE EFFECT OF BULKING ON YIELD AND STRENGTH OF CONCRETE

It is well known that nearly all of our laboratory tests, investigations and research on concrete have been based on mixtures with dry sand, while in practice on construction a dry sand is seldom available. Assuming that a 1:2:4 mix is under consideration, the laboratory bases its findings on the true mix with dry materials whereas on the job it is quite probable that the mix approximates more nearly 1:1.6:4. As a result of this the field concrete will have a richer mortar than intended, but there may be a deficiency of mortar which will affect both the workability and density of the concrete. If the workability is impaired, the tendency is to use more mixing water and thus the greater richness of the mortar may be offset to some extent at least. It follows, therefore, that a deficiency of mortar may have a direct effect on the strength and yield of concrete.

For the purpose of studying the effect on yield and strength 96 6 by 12 inch concrete cylinders were made and tested. The gradation of coarse aggregates was introduced as an additional variable in order to learn its effect also. The fine aggregate was Potomac

River sand and where damp sand was used it contained 6 per cent of water by weight. The materials were proportioned by volume under uniform conditions. The consistency was kept as constant as possible by the use of flow table measurements for each cylinder, in addition to measuring the amount of mixing water beforehand.

The cylinders were made in sets of 16 on six different days, a complete series being cast each day. The operators doing the mixing and molding were changed from day to day so as not to carry any one personal equation throughout the work. It was found that the measurements for yield and the results of the compressive strength tests showed substantially the same trend for each set of 16 cylinders. The average results of the tests on the 96 cylinders are shown in Figure 5.

CONCLUSIONS

The test results recorded in Figure 5 form the basis for the following definite conclusions:

1. With gravel concrete the yield per unit volume of coarse aggregate exceeds that with trap rock by about 6 per cent.
2. The decrease in yield due to the omission of coarse aggregate between one-fourth and three-fourths inches in size is from 3 to 4 per cent.
3. The decrease in yield due to using the nominal volume of damp sand without a correction for bulking is 7 per cent for a 1:2:4 mix and 8½ per cent for a 1:2:3 mix.
4. The decrease in yield due to using damp sand without a bulking correction and omitting the smaller sizes of coarse aggregate is 12 per cent.
5. For a 1:2:4 mix, the mixes made with dry sand averaged 14 per cent stronger than those made with the same volumes of damp sand. For a 1:2:3 mix, the mixes made with dry sand averaged 3 per cent weaker than those made with the same volumes of damp sand.
6. The omission of the smaller sizes of coarse aggregate resulted in a decrease in strength of from 12 to 14 per cent.
7. With a 1:2:4 mix, the cylinders made with dry sand and well-graded coarse aggregate averaged 17 per cent stronger than those made with damp sand and poorly graded coarse aggregate. With a 1:2:3 mix, this difference in strength was 12 per cent in the same direction.

It should be borne in mind that the above conclusions are based on laboratory specimens only and can not as yet be interpreted to apply directly to field conditions. In fact, it is not the intention of this article to advocate the practice of making a volumetric correction for the bulking of sand on the job until it has been demonstrated that the scheme for making the correction is practicable, also that as a result the concrete is more economical and at least as strong as under present practice. It is thought, however, that the additional sand introduced into the mix, where an adjustment is made for bulking, may tend to improve the workability of the concrete and thus assist in maintaining a uniform consistency and securing a more economical and uniform concrete. All of these possibilities are yet to be demonstrated under actual construction conditions.

(Continued from page 20.)

been reached for some of the specimens it was slowly released, and the recovery of the specimen noted. In some instances this amounted to almost one fourth inch. It thus becomes evident that when the concrete slides over the subbase there is a considerable amount of yielding of the base material, and it is therefore to be expected that if the material is fairly homogeneous, and the movement of the concrete takes place slowly the material of the subbase will gradually yield under this movement, and will thereby offer less resistance than these tests seem to indicate.

LOWERING FRICTIONAL RESISTANCE SHOULD REDUCE FREQUENCY OF TRANSVERSE CRACKS

In considering friction results, the maximum resistances are naturally of greatest importance, since these create the greatest stresses. Those given in the first table are therefore of more interest than those in the second, and these results very clearly show that the friction varies considerably, depending upon the character of the subbase. The indications are that friction can be greatly decreased if proper care is given to the preparation of the subbase. Every ridge or depression that is in the subbase surface before concrete is deposited furnishes an additional grip for the concrete, thereby tending to elastically deform a larger amount of base material, and thus offering greater resistance to the sliding of the concrete.

The formation of transverse cracks in the concrete bases can readily be ascribed to direct tension due to frictional resistance at a time when the concrete is contracting, whether this is caused by decrease in temperature, or by drying out of the moisture. The test results show that the coefficient of friction can readily vary from almost 0 to something over 2 or more depending upon the movement of the concrete and the character of the subbase. The distance between transverse cracks is dependent upon the coefficient of friction, and the total force of friction must extend over this distance. Calling the coefficient of friction f, the distance between cracks D, the weight of the pavement per square foot w, we may write the equation: $f \times w \times D =$ tensile strength of concrete per foot of width. Assuming the tensile strength of concrete to equal 100 pounds per square inch, the pavement to be 6 inches in thickness, the weight of a cubic foot of concrete to be 150 pounds, and the coefficient of friction to equal 0.5, the equation is stated as follows:

$$0.5 \times 75 \times D = 72 \times 100, \text{ or } D = 185 \text{ feet.}$$

If the coefficient of friction equals 2, the distance between cracks becomes 46 feet. It is conceivable that conditions may exist in which the frictional resistance may mount much higher than this, and may be excessive in spots.

It is quite apparent from the above formula that in order to reduce the frequency of transverse cracks, the subgrade must be made as smooth as possible and the concrete must be kept moist as long as practicable in order to prevent tensile stress from being set up in the concrete before it has acquired high resistance to tension.

BUREAU OF STANDARDS SCREEN SCALE FOR TESTING SIEVES

In the April issue attention was called to efforts being made by the American Society for Testing Materials to adopt a practical standard screen scale for testing sieves. Although progress is being made along this line by the society, the work of standardization is of necessity very slow. Until such standardization can be effected, the bureau believes that the United States standard sieve series, adopted by the United States Bureau of Standards, should be used wherever possible.

In a circular letter dated April 15, 1924, the Bureau of Standards calls attention to certain revisions in tolerances on wire diameter and average opening in its screen scale which make it possible easily to obtain sieves commercially which will conform to this standard. The Bureau of Standards also offers to test any sieve for the purpose of determining whether it conforms to the standard screen scale. The schedule of fees for this work is as follows:

Test and certification or report on any testing sieve	$2.00
Extra charge for placing a preliminary identification number on sieves not otherwise identified	.10
Test and report on a piece of sieve cloth, per square foot	3.00

Tests for the Federal Government and for State governments are conducted without charge.

Bureau of Standards screen scale for testing sieves

Sieve number	Sieve opening		Wire diameter		Tolerance in average opening	Tolerance in wire diameter	Tolerance in maximum opening
	Millimeters	*Inches*	*Millimeters*	*Inches*	*Per cent*	*Per cent*	*Per cent*
4	4.76	0.187	1.27	0.050	±3	−15 to +30	10
5	4.00	.157	1.12	.044	±3	−15 to +30	10
6	3.36	.132	1.02	.040	±3	−15 to +30	10
7	2.83	.111	.92	.036	±3	−15 to +30	10
8	2.38	.0937	.84	.0331	±3	−15 to +30	10
10	2.00	.0787	.76	.0299	±3	−15 to +30	10
12	1.68	.0661	.69	.0272	±3	−15 to +30	10
14	1.41	.0555	.61	.0240	±3	−15 to +30	10
16	1.19	.0469	.54	.0213	±3	−15 to +30	10
18	1.00	.0394	.48	.0189	±3	−15 to +30	10
20	.84	.0331	.42	.0165	±5	−15 to +30	25
25	.71	.0280	.37	.0146	±5	−15 to +30	25
30	.59	.0232	.33	.0130	±5	−15 to +30	25
35	.50	.0197	.29	.0114	±5	−15 to +30	25
40	.42	.0165	.25	.0098	±5	−15 to +30	25
45	.35	.0138	.22	.0087	±5	−15 to +30	25
50	.297	.0117	.188	.0074	±6	−15 to +35	40
60	.250	.0098	.162	.0064	±6	−15 to +35	40
70	.210	.0083	.140	.0055	±6	−15 to +35	40
80	.177	.0070	.119	.0047	±6	−15 to +35	40
100	.149	.0059	.102	.0040	±6	−15 to +35	40
120	.125	.0049	.086	.0034	±6	−15 to +35	40
140	.105	.0041	.074	.0029	±8	−15 to +35	60
170	.088	.0035	.063	.0025	±8	−15 to +35	60
200	.074	.0029	.053	.0021	±8	−15 to +35	60
230	.062	.0024	.046	.0018	±8	−15 to +35	90
270	.053	.0021	.041	.0016	±8	−15 to +35	90
325	.044	.0017	.036	.0014	±8	−15 to +35	90

The preceding table gives the standard sizes of screen wire and openings with allowable tolerances as recommended by the Bureau of Standards. A detailed explanation of this table is contained in Bureau of Standards Circular LC–74, copies of which may be obtained by addressing the United States Bureau of Standards, Washington, D. C.

BUREAU OF PUBLIC ROADS DISCONTINUES TESTING OF ROAD MATERIALS FOR GENERAL PUBLIC

On July 1, 1924, the bureau discontinued the general practice of testing samples of road materials for any citizen of the United States. Samples will be tested only when submitted by or at the request of a representative of the United States Government, or of a State, county, or municipality, and then only when accompanied by a statement to the effect that the sampling was done by a disinterested party. This statement should be made on United States Bureau of Public Roads Form T–206, and should include the name and title of the sampler.

INVESTIGATION OF STABILITY OF BITUMINOUS MIXTURES

Study of the factors which influence the stability of bituminous paving mixtures is being continued by the Bureau of Public Roads. Late in 1922 27 coarse-graded asphaltic concrete mixtures of widely different proportions were laid upon a circular track at the out-of-door laboratory of the bureau at Arlington, Va. These were subjected to truck traffic until their relative stability was shown by various degrees of displacement. Plans are now completed for the replacement of these first mixtures with a second series, 28 of which will be sheet asphalt and five will be coarse-graded asphaltic concrete. Certain indications developed by the original series have been utilized in the design of these mixtures and further information regarding the behavior of fine sand and coarse sand in comparison with the so-called sheet asphalt sand grading is expected to be brought out. Some of the sections will carry varying percentages of filler and in others asphalts of low susceptibility to temperature changes will be used.

It is manifestly impossible to subject to service tests all materials in the infinite variety of combinations to which they are susceptible. The ultimate objects of this investigation, therefore, are the development of the laws which influence stability and the evolution of a laboratory test, or series of tests, by which it will be possible to predetermine the service behavior and relative merits of bituminous mixtures. Consequently, in this scheme the service tests of a comparatively few mixtures of widely different composition and of known traffic-resisting characteristics are expected to play the role of a correlating agency between laboratory and field behavior.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JUNE 30, 1924

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1923			FISCAL YEAR 1924									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1923			PROJECTS COMPLETED SINCE JUNE 30, 1923			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 3,769,011.74	$ 1,794,703.21	367.9	$ 809,709.89	$ 391,544.33	96.2	$ 15,132,450.48	$ 7,400,051.40	866.4	$	$		$ 1,666,664.06	Alabama
Arizona	6,518,025.52	3,195,689.68	372.1	1,620,339.69	1,091,994.20	155.7	2,419,995.90	1,439,148.62	164.7	282,404.40	172,577.32	32.8	1,596,291.18	Arizona
Arkansas	8,623,205.72	3,308,572.12	736.2	2,471,545.59	1,115,773.51	208.2	6,247,130.36	1,986,982.36	342.9	1,618,539.66	654,010.77	81.1	1,997,061.24	Arkansas
California	6,435,547.60	3,538,967.90	340.5	4,503,527.43	2,106,180.27	193.2	11,871,819.13	6,395,695.08	434.1	1,092,308.63	499,050.87	25.2	4,551,411.88	California
Colorado	6,470,695.56	3,136,236.67	426.6	1,637,374.76	893,660.30	76.0	4,833,066.27	2,595,855.07	173.6	672,178.76	375,620.34	21.9	2,558,506.62	Colorado
Connecticut	2,568,643.95	1,026,148.85	60.7	494,226.07	243,409.75	12.9	3,041,169.37	1,029,509.90	62.3				1,082,126.50	Connecticut
Delaware	2,136,610.22	621,154.83	45.2	560,222.00	386,560.00	27.3	1,096,672.50	420,933.60	27.6	303,864.00	107,700.00	7.2	203,161.57	Delaware
Florida	69,466.31	29,700.63	16.6	691,667.76	431,770.29	33.2	9,190,175.12	4,506,550.50	259.4	398,642.71	171,123.94	10.3	1,147,741.64	Florida
Georgia	13,960,499.47	6,363,703.60	925.5	3,206,673.85	1,592,101.60	288.7	8,925,089.27	4,268,001.56	736.2	509,604.61	198,168.78	50.2	2,027,921.46	Georgia
Idaho	6,860,673.73	3,272,111.01	426.1	1,301,024.19	620,284.51	80.7	1,436,933.13	668,722.94	97.8	934,329.98	606,550.13	57.4	1,110,043.41	Idaho
Illinois	25,211,526.23	11,414,291.96	743.5	1,753,179.63	865,254.37	61.2	16,485,446.19	8,192,130.42	557.0	608,902.19	304,451.08	22.1	2,660,364.17	Illinois
Indiana	5,513,467.76	2,668,277.55	167.0	2,063,966.40	967,263.42	68.7	15,956,388.75	7,798,552.57	523.4	829,586.07	395,264.69	25.6	2,463,013.67	Indiana
Iowa	16,040,079.62	6,056,136.48	1069.4	7,165,696.57	3,160,895.38	623.5	8,663,469.34	4,026,657.08	622.9	289,642.44	144,342.39	42.5	1,928,105.67	Iowa
Kansas	12,216,428.90	4,246,436.34	349.7	4,867,707.58	1,734,738.46	153.0	16,669,331.36	6,248,427.04	596.0	2,874,528.15	1,196,606.53	121.6	1,611,076.63	Kansas
Kentucky	6,620,288.80	2,789,991.16	285.0	4,202,690.51	1,823,956.13	144.4	7,573,552.00	3,663,928.55	329.2	101,082.14	50,541.07	4.0	2,043,322.10	Kentucky
Louisiana	7,131,394.90	3,157,128.03	690.4	1,357,066.28	479,015.33	70.8	5,683,581.53	2,815,058.90	372.3				814,239.74	Louisiana
Maine	4,662,854.19	2,242,963.03	159.8	2,248,204.59	1,056,972.35	70.9	1,437,207.97	714,171.72	54.6	735,610.48	359,684.86	30.2	716,180.05	Maine
Maryland	5,309,394.94	2,524,543.54	185.0	1,450,649.48	686,478.24	58.2	2,225,748.35	894,686.95	71.0	435,565.02	138,232.09	12.6	402,710.18	Maryland
Massachusetts	6,841,975.16	2,802,967.88	163.7	3,349,226.86	1,302,769.34	69.1	5,173,074.47	1,800,430.47	90.4	475,458.32	142,847.12	7.1	1,670,775.19	Massachusetts
Michigan	9,526,041.53	4,343,617.14	363.4	3,906,093.54	1,716,796.09	131.1	13,609,397.79	6,587,382.19	484.6	169,111.25	80,165.50	6.2	3,161,592.08	Michigan
Minnesota	19,826,671.37	8,004,277.87	1893.4	4,208,889.87	1,881,565.20	398.6	8,600,577.00	3,623,184.40	676.4	671,124.09	290,100.00	113.7	1,519,291.53	Minnesota
Mississippi	4,746,771.31	2,284,557.86	462.4	3,141,422.68	1,544,383.53	192.6	7,695,258.21	3,839,698.61	489.8	313,302.99	133,316.80	10.0	1,729,316.20	Mississippi
Missouri	5,272,757.06	2,358,848.60	365.6	6,079,270.62	2,887,050.68	437.9	14,814,141.06	7,070,576.64	601.9	2,962,047.64	1,440,646.30	161.2	4,163,066.88	Missouri
Montana	7,935,185.30	3,896,053.50	711.5	932,093.86	488,281.62	79.9	2,537,636.91	1,922,348.47	265.6	1,396,549.47	760,776.09	119.6	3,876,957.32	Montana
Nebraska	3,159,814.47	1,449,675.04	475.6	4,716,522.69	2,266,016.55	964.8	6,675,996.20	3,244,725.73	687.2	1,133,884.44	561,762.35	100.2	3,929,766.33	Nebraska
Nevada	3,049,109.29	1,562,111.35	203.6	411,136.23	291,513.63	22.0	4,765,024.76	4,063,643.23	394.1	67,265.43	56,066.90	3.5	916,963.69	Nevada
New Hampshire	2,336,997.00	1,129,004.69	140.5	739,753.19	358,862.69	30.8	886,516.84	414,374.33	29.8	275,923.24	130,442.38	9.4	402,279.71	New Hampshire
New Jersey	5,423,362.96	1,975,783.52	112.1	2,200,432.16	665,747.97	36.6	10,226,184.47	2,728,243.65	70.1	417,573.68	107,667.49	4.1	1,091,804.37	New Jersey
New Mexico	4,706,909.12	2,433,663.68	617.1	699,377.33	325,186.00	97.2	6,165,553.69	4,041,486.28	657.7	825,138.02	507,459.84	54.7	1,281,536.20	New Mexico
New York	9,826,633.82	4,358,659.70	290.6	9,036,106.67	3,899,164.74	282.1	24,663,366.96	9,526,032.97	663.2	7,272,400.00	2,430,398.75	147.6	6,493,871.84	New York
North Carolina	11,770,363.76	5,325,763.37	826.8	797,369.21	350,994.29	57.9	9,623,694.36	3,484,417.74	266.6	2,364,959.10	1,060,134.58	66.2	2,072,941.02	North Carolina
North Dakota	5,142,167.55	2,492,599.09	792.5	3,946,815.56	1,925,906.33	795.4	3,159,113.69	1,554,191.41	504.0	518,237.30	259,119.11	80.8	2,131,840.06	North Dakota
Ohio	22,840,050.09	7,966,325.17	662.6	10,282,701.34	3,913,592.82	299.7	12,769,366.01	4,853,124.66	361.1	1,674,801.96	529,493.47	42.4	2,677,627.88	Ohio
Oklahoma	9,795,634.54	4,417,788.46	371.5	3,191,230.72	1,471,063.57	125.8	8,689,043.76	4,194,467.46	389.8	2,008,286.36	826,077.47	82.4	1,628,306.04	Oklahoma
Oregon	10,391,362.48	4,782,736.36	534.1	1,691,510.69	1,036,357.41	121.5	2,927,450.56	1,706,203.60	175.6	486,601.30	298,714.84	29.5	680,146.77	Oregon
Pennsylvania	33,090,029.45	12,615,056.79	652.6	3,735,219.53	1,499,636.00	76.9	15,325,663.77	4,613,831.25	268.5	7,246,308.80	1,820,961.26	118.7	4,052,138.71	Pennsylvania
Rhode Island	1,484,075.35	647,634.96	38.6	290,321.90	131,593.00	7.4	1,067,787.06	411,769.49	22.7	334,265.25	89,147.31	5.6	652,906.24	Rhode Island
South Carolina	6,161,485.41	2,885,754.37	665.1	2,834,991.32	1,238,290.86	269.3	5,998,020.81	2,360,714.93	517.6	547,844.28	162,127.61	28.1	1,040,668.24	South Carolina
South Dakota	4,756,907.24	2,320,007.33	534.5	3,915,690.62	1,924,628.94	455.3	6,268,470.80	3,166,316.97	793.1	674,713.27	306,887.96	38.4	980,836.80	South Dakota
Tennessee	3,210,970.94	1,535,762.16	112.5	3,594,712.41	1,775,173.89	147.1	13,761,114.87	6,501,844.69	433.3	1,333,366.80	580,307.44	52.6	1,628,548.80	Tennessee
Texas	29,172,501.06	11,094,069.77	2259.6	13,169,497.50	5,096,555.14	863.2	25,874,603.79	10,200,655.43	1624.6	2,875,326.43	1,039,200.03	165.7	4,293,732.63	Texas
Utah	2,175,007.41	1,196,437.68	116.6	1,129,416.34	699,368.24	102.4	4,382,736.86	2,833,446.16	304.2	351,526.03	250,895.55	28.6	1,136,325.38	Utah
Vermont	1,370,715.63	669,433.75	53.1	551,396.53	273,335.37	21.3	1,527,160.47	731,432.50	42.6	357,120.31	176,795.65	7.9	682,981.73	Vermont
Virginia	6,823,978.36	3,279,029.56	402.5	3,211,323.12	1,522,752.67	160.0	8,859,252.86	3,975,621.01	302.7	1,110,631.09	183,718.10	35.1	1,631,631.46	Virginia
Washington	10,356,960.62	4,618,105.70	419.8	1,027,634.85	471,789.75	37.2	2,861,003.81	1,306,000.00	121.6	326,467.31	140,000.00	9.2	1,149,782.65	Washington
West Virginia	4,100,225.73	1,800,932.41	206.3	1,389,522.22	564,109.12	49.3	4,823,816.81	2,168,539.55	163.8	822,524.01	371,845.22	28.2	856,705.70	West Virginia
Wisconsin	15,564,691.39	5,996,989.24	1077.6	3,249,211.76	1,444,044.33	247.7	3,546,039.10	1,763,705.92	150.4	237,619.47	106,939.26	10.4	4,366,772.25	Wisconsin
Wyoming	4,601,439.36	2,210,774.20	534.8	1,526,166.25	867,324.50	152.8	4,365,613.36	2,647,849.96	327.4				961,402.36	Wyoming
Hawaii														Hawaii
TOTALS	$ 407,704,641.14	$ 174,044,673.82	23297.2	$ 141,950,750.13	$ 63,807,726.00	9155.7	$ 360,649,591.14	$ 172,613,311.94	18053.5	$ 50,927,944.87	$ 20,236,993.42	2119.7	$ 365,625.00 / $ 94,422,294.82	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 55,507,667.40 Federal aid $ 25,385,774.44 Miles 2704.0

APPORTIONMENT OF THE FEDERAL AID FUND

FISCAL YEAR 1925

States	Amount	States	Amount
Alabama	$1, 542, 052. 56	Nebraska	$1, 577, 155. 34
Arizona	1, 053, 003. 56	Nevada	947, 623. 25
Arkansas	1, 258, 857. 07	New Hampshire	365, 625. 01
California	2, 464, 990. 78	New Jersey	936, 413. 03
Colorado	1, 361, 482. 06	New Mexico	1, 185, 528. 88
Connecticut	475, 513. 91	New York	3, 663, 105. 86
Delaware	365, 625. 17	North Carolina	1, 697, 246. 16
Florida	887, 336. 52	North Dakota	1, 178, 708. 13
Georgia	1, 983, 022. 99	Ohio	2, 795, 804. 69
Hawaii	365, 625. 00	Oklahoma	1, 753, 189. 71
Idaho	936, 698. 01	Oregon	1, 176, 830. 15
Illinois	3, 203, 867. 99	Pennsylvania	3, 365, 956. 21
Indiana	1, 939, 903. 32	Rhode Island	365, 624. 87
Iowa	2, 078, 248. 33	South Carolina	1, 054, 028. 17
Kansas	2, 081, 230. 04	South Dakota	1, 209, 144. 18
Kentucky	1, 411, 584. 45	Tennessee	1, 628, 740. 97
Louisiana	995, 301. 59	Texas	4, 410, 169. 76
Maine	686, 453. 36	Utah	847, 741. 90
Maryland	635, 945. 01	Vermont	365, 625. 27
Massachusetts	1, 089, 806. 22	Virginia	1, 448, 562. 55
Michigan	2, 226, 824. 73	Washington	1, 113, 308. 17
Minnesota	2, 120, 906. 56	West Virginia	798, 275. 47
Mississippi	1, 294, 371. 65	Wisconsin	1, 877, 600. 32
Missouri	2, 423, 485. 75	Wyoming	936, 372. 13
Montana	1, 544, 483. 19	Total	73, 125, 000. 00

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 6 ▽ AUGUST, 1924

THE LACKAWANNA TRAIL IN PENNSYLVANIA—AS SAFE AS A ROAD CAN BE

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 5, NO. 6 AUGUST, 1924 H. S. FAIRBANK, Editor

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THE U. S. BUREAU OF PUBLIC ROADS

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TRAFFIC CONTROL AND SAFETY

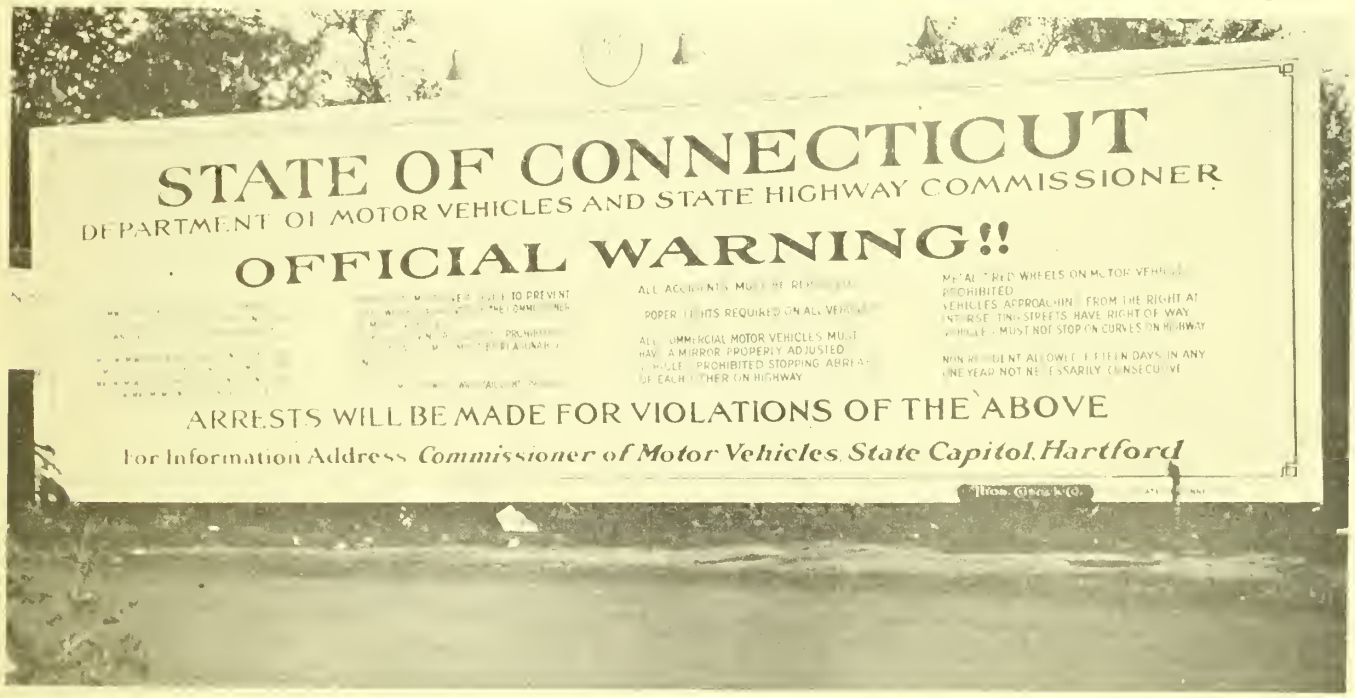
A REVIEW OF EXISTING METHODS AND DEVICES FOR PROMOTING HIGHWAY SAFETY

By E. W. JAMES, Chief, Division of Design, U. S. Bureau of Public Roads

THE construction, maintenance, and administration of public highways has become the largest element in the program of public works in the United States. A density of traffic represented by a registration of fifteen and a quarter million motor vehicles in the United States presents a new administrative problem. The annual increase in traffic accidents with growing fatality to life and increasing loss and damage to property emphasizes the immediate and imperative need of effective steps toward traffic control and greater consequent safety to users of the road. The rapid production of additional improved mileage, the closing of open links, the betterment of bad sections and the resultant extension of long, unbroken routes of good roads, with the accompanying inevitable growth of long distance travel and touring by automobile, will certainly cause a corresponding increase in the danger attending highway travel.

This article will outline briefly the several aspects of the problem and indicate in general terms the advance so far made in the direction of traffic control and safety.

There are four fundamental considerations. First, there is the independent safety of the individual driver; then the common safety of passing drivers. Next is the question of safety for pedestrians, and finally there must be due consideration of the economic maintenance, care, and preservation of the road itself. No solution of the matter will be complete that fails to give fair consideration to any of these phases of the problem. The lines between them are not sharply drawn. Details of design and regulations accomplishing a satisfactory end from the point of view of the individual driver may offer at once an equally effective solution of the problems raised by passing traffic, and may serve also to protect the pedestrian. Control of traffic on behalf of the road surface will likewise promote the general safety of traffic by reducing speeds, widths, weights of vehicles and by restricting and regulating other elements likely to affect it unfavorably.


STATE OF CONNECTICUT
DEPARTMENT OF MOTOR VEHICLES AND STATE HIGHWAY COMMISSIONER
OFFICIAL WARNING!!
ARRESTS WILL BE MADE FOR VIOLATIONS OF THE ABOVE
For Information Address Commissioner of Motor Vehicles, State Capitol, Hartford

Signs giving the most important highway regulations are erected on all of the main roads entering Connecticut

BASIC PRINCIPLES OF HIGHWAY SAFETY

Recognizing these four considerations as the ends to be attained, the difficulties of the problem present themselves in matters of detail, expediency and practicability rather than in the establishment of fundamentals. The basic principles are easily defined and generally accepted. Reduced to the simplest terms they may be stated as follows:

1. The development of safe roads by elimination of all known dangers in so far as such elimination is possible within physical and economic limitations.

Although the problem is comparatively new it has already received considerable attention. Many, if not all, of the States have enacted laws relating to traffic control in one form or another and several highway departments have developed a systematic practice for increasing the safety of their highways. Some States, as for instance Maryland, Connecticut, and Pennsylvania, have made systematic surveys of traffic conditions and have altered their practice in design according to the indications of such data. The American Association of State Highway Officials have a special committee now at work studying the whole subject, and the Bureau of Public Roads has gathered considerable data relating to safety devices, traffic control, and details of design calculated to increase safety on the highways.

2. Warning of uneliminated dangers by means of easily recognized signs and other devices of standardized form, uniformly placed with respect to the danger.

3. The development of safe vehicles by perfection of running parts and adoption of safety devices.

4. Diligence and care in the licensing of drivers of motor vehicles to eliminate the incompetent, careless and irresponsible driver.

5. The development of simple, uniform, and effectual traffic regulations and the education of the whole public to a complete acquaintance with them.

Sign erected by Maryland State Roads Commission at top of a long, steep grade.

6. The rigid enforcement of traffic regulations.

There is little doubt that a solution of the problem worked out on the basis of these general principles would have the effect of reducing the dangers of the open road to a minimum. It is the means of applying them practically and to the best advantage and especially the harmonizing of action by the 48 States that remains to be worked out. Already there have been applied in several of the States adequate solutions of some details of the problem, which if adopted generally would go far toward meeting the situation satisfactorily. One State has danger signs which are difficult to improve upon. Another has a scheme of road surface markings of decided merit. Our cities have developed by trial and experience excellent rules of the road. Some States and many cities require evidence of competence before licensing drivers. What needs to be done first, and what is already being done by the Committee on Traffic Control and Safety of the American Association of State Highway Officials which is working on the problem, is to review carefully the practices current in the several States. By so doing it should require but comparatively little effort and adjustment to secure the adoption of uniform measures by all States, which will take us far toward the desired solution of the highway safety problem.

INVESTIGATION OF CAUSES OF ACCIDENTS AN IMPORTANT PRELIMINARY STEP

As a preliminary step it is important that a careful study be made of the causes of accidents occurring on the highways. Studies of this sort have already been made in several States, and have proved invaluable as a guide to State officials by indicating the relative danger of various practices and conditions requiring correction. Such a record kept by C. H. Purcell, District Engineer, United States Bureau of Public Roads, in Oregon and Washington, rates the various causes of accidents occurring in those States during December, 1923, as follows:

	Oregon	Washington
Time:	*Per cent*	*Per cent*
Daylight	41.5	46.4
Darkness	53.6	44.9
Undetermined	4.9	8.7
Nature of accident:		
Collision between two motor cars	41.5	36.2
Collision between motor car and train	7.3	1.5
Motor car leaves road	36.6	43.5
Equestrian or pedestrian run down	12.2	13.0
Undetermined	2.4	5.8
Location:		
Grade crossing	7.3	2.9
Intersecting highways	12.2	7.2
On tangent	29.3	17.4
On curve	12.1	14.5
On grade	9.8	11.6
Undetermined	29.3	46.4
Visibility:		
Raining	2.4	
Snowing	2.4	2.9
Foggy	7.3	1.5
Clear	17.5	2.9
Undetermined	70.4	92.7
Causes:		
Faulty operation by driver—		
Incompetence and inexperience	7.3	2.9
Operation by intoxicated persons	7.3	4.3
Excessive speed	4.9	4.3
Recklessness and carelessness	51.3	59.4
Violation of traffic rules	7.3	14.5
Faults of other than drivers—		
Pedestrian fails to give right of way	(1)	1.5
Miscellaneous	(1)	5.8
Faulty equipment—		
Glaring headlights	(1)	5.8
Mechanical breakdowns	(1)	1.5
Faulty highway conditions	None.	None

[1] Undetermined.

A roadside mound taking the place of a guard rail on a Massachusetts road

This record may not be in the best form to adapt it to comparison with other similar records which have been made in other States, notably in Maryland and Connecticut, but it appears to check those records in some interesting details. It would be an easy and simple matter to secure analyses of accidents in some uniform way wherever such analyses are attempted; and such records from widely separated sections of the country, by their agreement as to the relative hazards of the various causes of accidents, would point clearly to the corrective measures most urgently required and indicate the fields in which the work of standardization will be most effective in reducing dangers.

AVOIDANCE OF DANGEROUS ROAD CONDITIONS REQUIRES CONSTANT WATCHFULNESS

Much remains to be done in developing practice in design and construction of highways that will reduce the danger of traveling the open road. In this respect, as in others, the sources of danger are comparatively well known. What is needed is constant watchful-

ness to prevent the perpetuation of dangerous conditions, an aggressive policy of safety promotion, and public support for the expenditures required, which, in some cases are quite heavy.

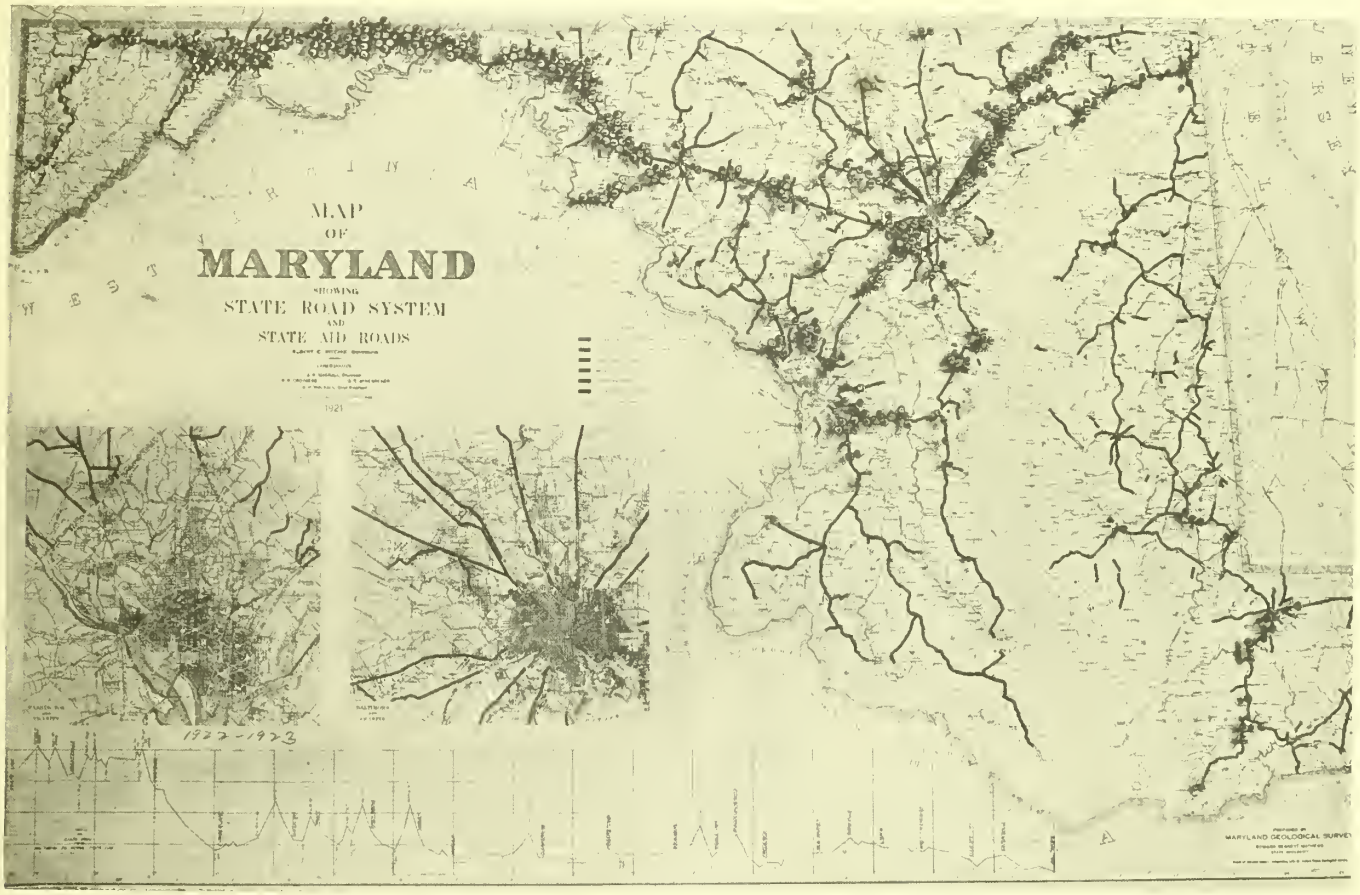
Few persons, asked to catalogue the principal dangers of the road, would fail to include those in the following list:

Blind curves and road intersections
Sharp curves on embankments
Unprotected embankments
Narrow bridges
Sharp convex vertical curves
Slippery road surfaces.
Steep grades
Narrow road surfaces
Low or rough shoulders
Steep crowns
Sharp curves at bridge and underpass approaches
Grade crossings
Unsuperelevated curves

rapidly increasing traffic has forced the State to reconstruct a large number of curves on their earlier-built roads to make them safe. This practice has prevailed also for many years in Massachusetts.

18 FEET THE MINIMUM SAFE TWO-WAY WIDTH

Massachusetts formerly built a 14-foot macadam as its standard design. Maryland now frequently uses a 15-foot cross section even on new work. But in general, the opinion of engineers is crystallizing on 18 feet as the minimum safe two-way road surface, and in many cases a 20-foot standard section is being used on the principal routes. Safety against being forced to the shoulders depends first on ample width of surface and



Maryland's accident map has been useful in discovering the places on her State road system in need of betterment to relieve danger. Note the clustering of pins in mountainous western Maryland

These, then, are some of the defects which should be avoided, so far as practicable, in future construction. A most important detail is that of providing adequate width for traffic and controlling the width of vehicles, because a hazard exists to passing drivers in having to leave the road surface on account of the inconsiderate or incautious driving of the other man. This danger is not confined to curves but is as great perhaps on tangents if we are to believe the evidence of the Maryland and Connecticut records. Against this, danger we can employ correctives in design and many States are doing so, especially on curves. The widening and superelevation of curves is now becoming very common and is standard practice on all new work in many States. There exists considerable variety in the assumptions and details of design, but the defects are essentially alike in all cases. In Maryland the on restriction of the width of vehicles. This latter must be a matter of traffic law and regulation, and the prevailing gauge of vehicles now used for generations indicates reasonable requirements to be enforced without great difficulty.

There always exists some danger of accident to passing vehicles because of loss of control by one or the other driver. Sometimes this is "psychological," due to nervousness or inexperience, inability to gauge correctly the clearance of the vehicle or the state of mind of the other fellow. This hazard is reduced by widening the road surface, and may always be further reduced by the control of speed and the reduction of gradients. "Speeding has no excuse" is a maxim that should be impressed on every driver. The world lived and progressed when limited to a speed of about 4 miles on the open road, and 18 or 24 miles give ample

multiple over the old way. Speeds in excess of 25 miles per hour are always likely to be dangerous because of the unforeseen.

On excessive gradients some States now warn the unsuspecting driver to attempt the down grade only in second gear. This indicates the element of competence and experience, of good judgment and care that must in the last analysis be depended upon in the critical case. The determination of the minus per cent of gradient requiring use of second gear in the average car, if it is to be a safe grade to travel, should be made, and a standard warning sign adopted.

In safe design we must emphasize, then, adequate original width of surface, control of width of vehicle, flattening and strengthening of shoulders, the use of guard-rail and other protective structures where high fills require them, the superelevation of curves and the lengthening of convex vertical curves, the securing of sufficient line of sight, the proper adjustment of profile and alignment to avoid dangerous combinations of gradient and curvature, and the proper type of surface.

Maryland's way of lessening danger at narrow bridges.

FEDERAL AID FOR ROADS A HARMONIZING FORCE

The Bureau of Public Roads has done much through its approval of designs for all Federal-aid roads to harmonize and develop good practice with respect to all the details mentioned. It has neither authority nor disposition to force matters, but it has waged an assiduous campaign of education and given the strongest possible support to all the State highway departments in securing all kinds of betterments in design. In States where the legal highway is along the section line, the flattening of curves has become common where it was unusual, grades are being reduced, guard rail included in designs, bridges widened and strengthened, and railroad grade crossings eliminated. In the enormously expensive and important item of grade crossing elimination alone there has been remarkable advance. No less than 25 per cent of all crossings so far encountered in the Federal-aid system have been eliminated by relocation or separation of grades in the premises. At present no standards of curvature or gradient are prescribed, because conditions are of such great variety throughout the country; but tentative policies have gradually hardened into more or less fixed practice that is generally followed. Some States, as for instance North Carolina, have adopted a maximum gradient of 6 per cent or lower, in line with the most advanced advocates of grade reduction. Other States, having a very irregular topography and advanced development, have not been so successful.

It is easily foreseen that there will be necessary in future years much betterment and replacement on highways already well located and improved. The banking and widening of curves, the reconstruction of low-capacity, narrow bridges, the flattening of shoulders and widening of fills, and the extension of the line of sight at curves will have to be done at considerable cost, because in the original design features were retained which are causes of accidents.

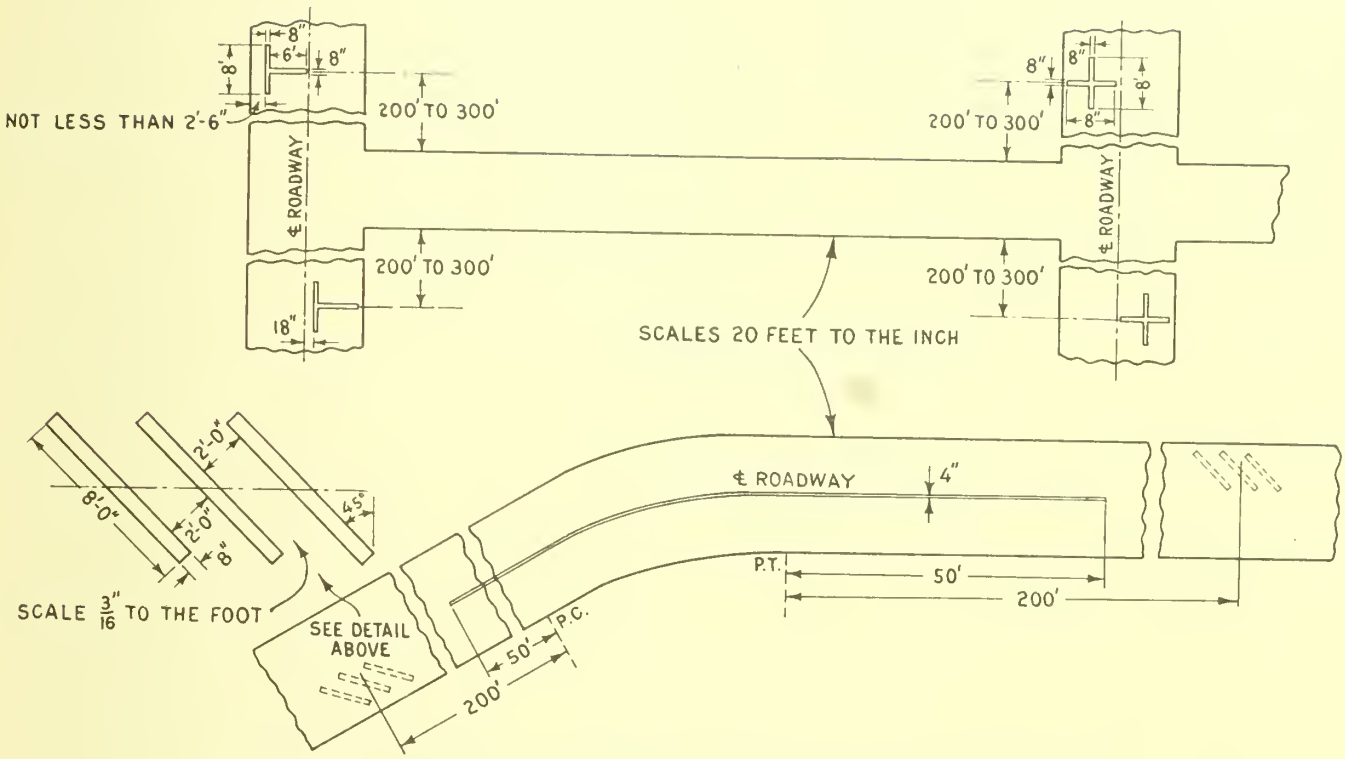
An excellent device which serves to draw attention to these danger spots in existing roads is an accident map such as that used by the Maryland State Roads Commission and others for several years. Adding a pin to such a map of the road system at the location of each accident as it occurs, and distinguishing between the causes of accidents by means of vari-colored pin heads serves to attract attention to the more dangerous places on the roads by the inevitable clustering of the pins. By carefully watching the map the Maryland Commission has been able to correct many dangerous conditions, and the beneficial results show up immediately in the subsequent map record.

In some instances the danger revealed by the map can only be dealt with by reconstruction or betterment of the road; in others the placing of a danger sign is the only practicable remedy at the time, and often does much to relieve the danger. Examples of such solutions in Maryland are shown in the illustration of the mountain grade warning sign on page 2 and the one-way bridge sign placed at the approach to a narrow bridge which, for the time being, it is impracticable to replace, as shown on this page.

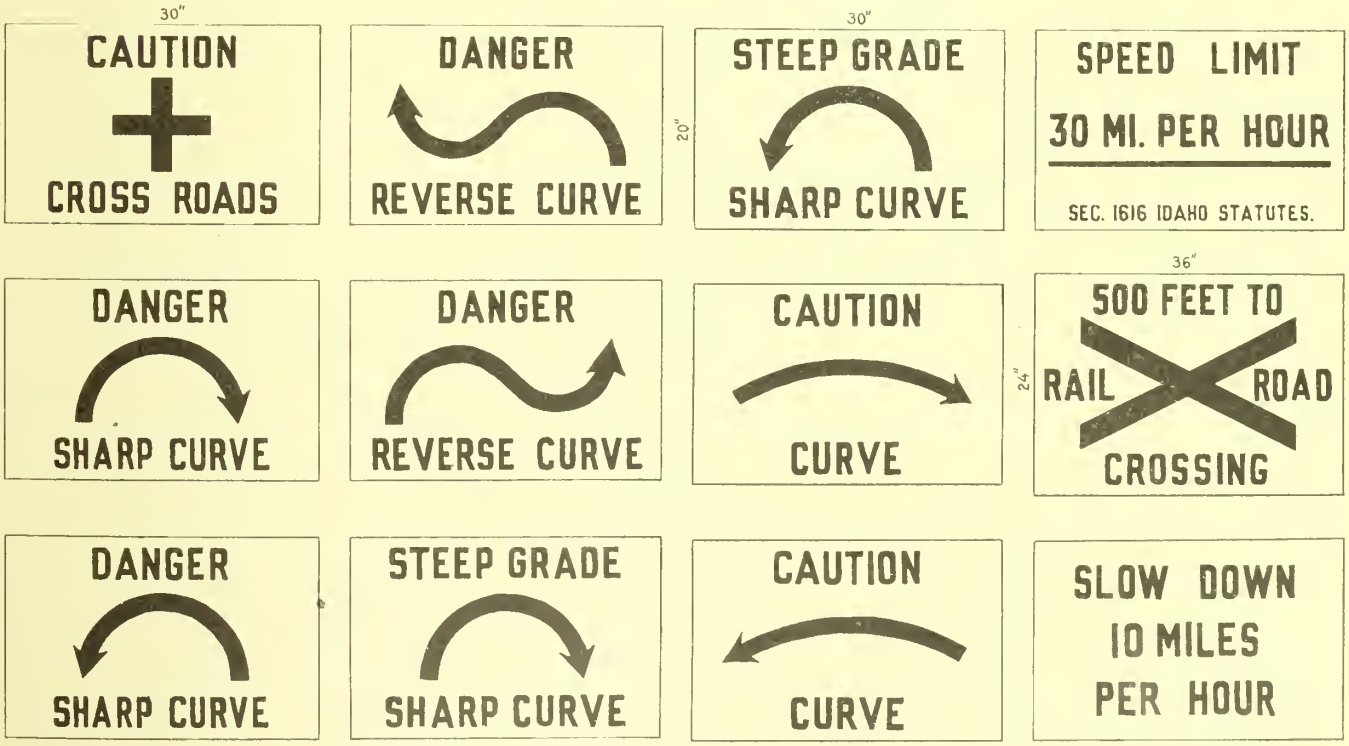
UNIFORMITY IN DANGER SIGNS LARGELY A PROBLEM OF SELECTION

It is, of course, impossible to build a highway which will be absolutely free of danger to its users. Curves and road intersections, grades, and embankments must remain in the best of roads, and each is potentially dangerous. All that can be done, then, is to give the traveler due warning of the danger. Signs, either diagrammatic or worded, are commonly used for this purpose, and in some States these have reached an entirely practical and useful stage of development. For instance, the series of danger signs adopted by the State of Idaho is especially effective. They are uniform in design, simple, bold, and striking. In contrast with these are those which rely too much on wording and require too much attention on the part of the passing driver. Washington, New Jersey, Ohio, and Tennessee have already adopted schemes essentially like that of Idaho and the possibilities of standardization are many, some of them obvious. Drivers in different States will recognize the signs as having identical meaning, the effect of repetition will educate, the use of the sign will become more general, the cost of manufacture will be reduced, the method of installing can be made more uniform than is now possible, and this in turn will expedite and cheapen the installation. The development of a complete series of danger signs and their general adoption and use is a problem in the safety of our highways.

Another method of reducing hazard at curves is by use of markings on the road surface to direct the path of traffic. The commonest mark is the "center line" in white or black. The latter, first adopted, is now generally giving away to white, which has much greater visibility under prevailing conditions. The system of traffic markings has been most completely developed probably in Massachusetts, but is also used generously



In Massachusetts signs painted on the surface of the roadway give warning of curves and intersecting highways



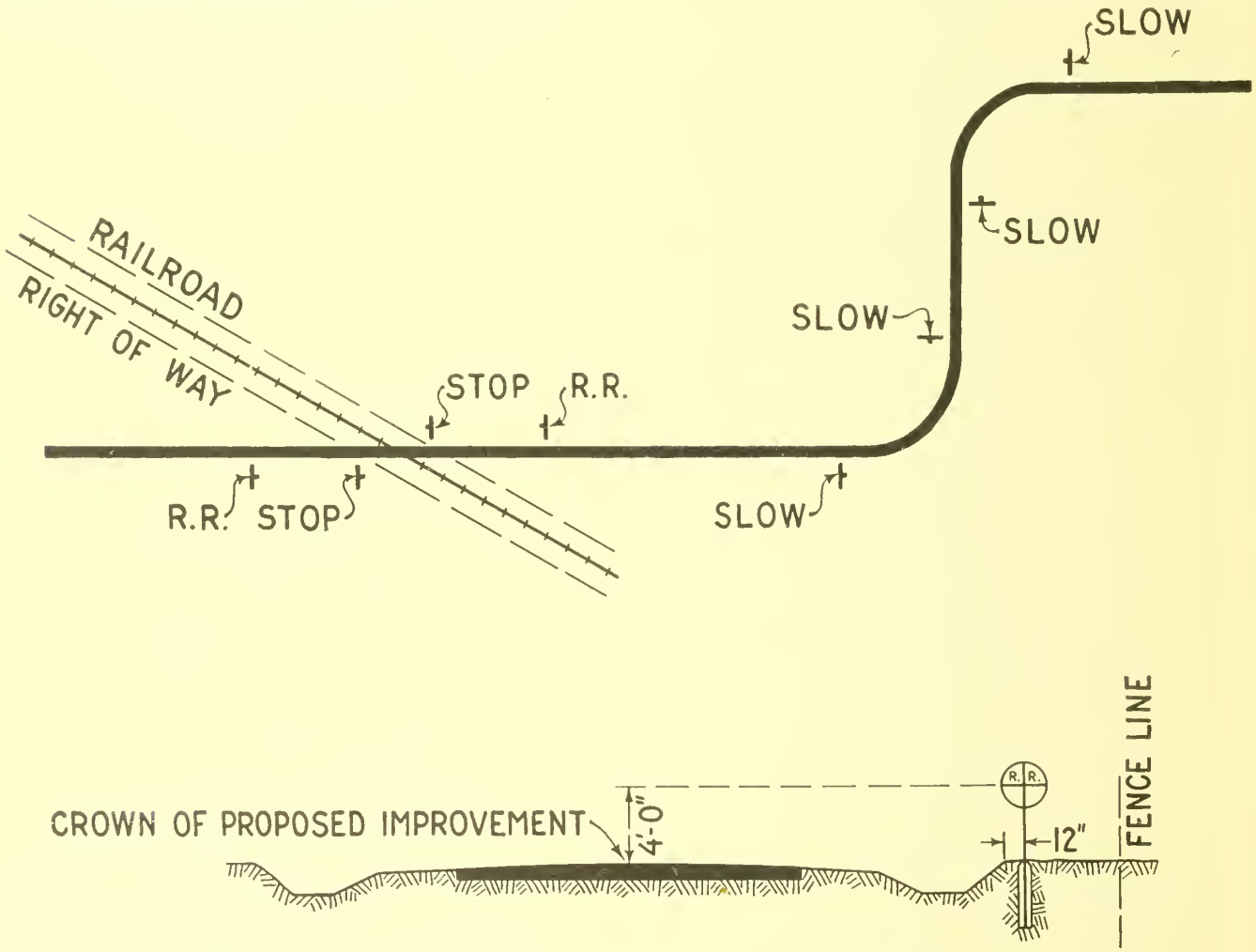
Standard danger signs adopted by the Idaho Department of Public Works

in Pennsylvania, Maryland, Delaware, New York, Illinois, and other States. This system is effectually confined to "black top" pavements, concrete, and other hard surfaces. In Pennsylvania experiments have been made in constructing a white center line integral with a concrete pavement. This is expected to save the cost of maintaining such markers and also to make it possible to extend their use widely—to tangents as well as to curves.

The diagrams on pages 5 and 6 of the location of signs and surface markers show the relative position of these with respect to curves and intersections. These are taken from Massachusetts and Kansas practice.

glance the nature of the hazard, he is prepared to meet it with due caution. Another element in promoting the safe use of the highways is indicated in the development and extension of these surface markings; and in this connection attention should be called to the great educational effect that is already produced by the common use of such markings on city streets. Drivers are learning to observe them.

Although accident records consistently indicate a relatively low percentage of wrecks due to faulty equipment, consideration should be given to certain details of motor-car design that are now largely responsible for the worst accidents and severest injuries occurring on



Plan for the placing of warning signs at grade crossings and dangerous curves used by the Kansas Highway Commission

CENTER LINES EFFECTIVE

The strongest effect produced by the center line at horizontal and vertical curves is its mental impression on the driver. Everyone knows that if a collision occurs, the blame rests on the one who is off-side. The center lines on blind vertical curves, and on curves in alignment of low visibility are powerful correctives of recklessness; they impress the observance of rules of the road, and obviously by so doing promote safety.

Surface markings are used to indicate highway intersections and railroad grade crossings as well as curves. The driver watching the road sees these as quickly and certainly, or perhaps more certainly than he does signs at eye level alongside of the road, and as the markings devised in Massachusetts, for instance, explain at a

account of defects in equipment. Probably the most conspicuous of the faulty details is the presence and use of glaring headlights and spot lights. This is a matter that is susceptible to correction by regulation, and it may be possible to apply the regulation at the source and actually prevent the manufacture and sale of those types of lamps that are most dangerous, as well as to prevent their use on vehicles.

WHAT OF FOUR-WHEEL BRAKES?

Next to glaring lights it is probable that defective brakes are the most frequent causes of collision attributable to equipment. This emphasizes a fact that is noted in connection with a large percentage of accidents involving mechanical devices. The very means of

safety to which we trust ourselves fail to act. The lesson is one for the car owner and no amount of regulation or inspection is practicable that would effect a remedy in this case. The man who undertakes to operate a power vehicle on the public highway at speeds approaching or exceeding those of many railroad trains, but without the safe guidance of a track, must depend to so large an extent on the effectiveness of his brakes that he should assume the responsibility of this detail at least. And if he fails to assume it and has a collision in consequence he should be held accountable beyond question. Faulty brakes should never be accepted as an excuse for anything.

The attempt of manufacturers to increase the security of braking devices by adding them to the front wheels may reduce the chances of accident by doubling the brakes available, or it may increase accidents by instilling a too great confidence in brakes. However, the introduction of the four-wheel brakes serves to indicate that the question of providing greater security of equipment is receiving consideration by those concerned in its manufacture.

The introduction of balloon tires also has its effect on the ease with which a car is steered and on the tractive effort exerted on the road surface. These tires doubtless do less damage to road surfaces than harder pneumatics or solid tires, but they may somewhat increase the stiffness of the steering mechanism.

The use of nonsplintering glass, automatic stop lights, reflecting mirrors, cowl and fender lights indicate some of the lesser details that aid in the reduction of accident or injury.

Other features of design, affecting more particularly the heavier and larger motor equipment, must be controlled in the interest of general safety. The width of truck body or of load must be kept down to a safe dimension and should not be greater than that customary for a standard guage vehicle with ordinary extensions such as wheel guards or fenders. A vehicle so wide that it converts a two-way road for a moment in passing into a one-way road so that traffic in the opposite direction is forced to the shoulder is never anything but a hazard, and what is more it represents license, not liberty, in the use of the public highway.

Consideration of the safety of the traffic will not produce a sufficient solution of traffic perils. There is the road itself to be considered. The relegation of a once substantial and serviceable method of road building to the discard of old practice was the direct result of two items—speed and tractive resistance. Road surfaces are still the victim of speed and to some extent of tractive resistance.

The control of weights is essential to the economic upkeep and reasonable life of the road itself and should be exercised through adequate regulations.

The idea exists still that the highway is an 18-foot track upon which any vehicle may be run that can be got upon the surface. Not so long ago the writer heard a high official of a prominent manufacturing company declare that the sound and economic procedure is to build the most economic truck or car possible, design it of such width, height, weight, and speed as is mechanically soundest and leave it to the highway administration to produce the modern highway required as a track for such an ideal vehicle. This way lies highway suicide. The public will pay the bill and no second mile of such highway would ever be built if a full discussion of the first mile were made public.

Safety of the road itself depends upon control of speed, of load per unit of bearing area, of width and height of body, of type of vehicle and of tire. The State of New Jersey has a traffic-control law considered as a model. It probably represents the maximum permissible requirements in most of its details and future developments will be toward lower requirements. Studies of actual traffic indicate this.

NO SOLUTION EFFECTIVE WITHOUT COMPETENT DRIVERS

The safety of the highways will not, however, be entirely secure until there is a standard of competence fixed as a prerequisite to driving. The wild driver is still at large on our highway and no details of design will save him from accidents. He may be a simple speed maniac or he may be a hopelessly erratic driver. He must be left to the tender mercies of the law, which should be executed not in consideration of him, but solely in consideration of the other fellow.

It was formerly believed that accidents prevailed at curves, either of grade or alignment, but there is considerable evidence from data compiled in Maryland that greater dangers of colliding exist, as cars are now driven, on the long, straight sections where the drivers "step on it" to the point of recklessness. However, such results are to be overcome only by the education of the man, not by alterations in design. The fact remains that curves and highway intersections are points of potential danger and become more than usually safe only because a driver is impressed with the exisiting hazard and avoids it.

It becomes at once important therefore that drivers be impressed with their responsibility to the general public to the point, first, of being allowed to drive only when proficient, and in the second place of driving strictly in accordance with the law, the rules of the road, and common courtesy. The pedestrian at least is interested to a certain degree in the last.

His rights are being infringed to a certain extent on all our highways, not so much by reason of his being excluded from the road, but because he can not use the road without exposing himself to hazards more or less serious. The rule of the sea in respect to sailing ships is an old and tried one, safe, and sane, and reasonable. The steamer shifts her course to protect the sailing vessel. The vehicle of greater power and speed yields to the weaker and slower. So the automobile should yield to the pedestrian when their paths intersect. To avoid annoying and frequent intersections they both should observe common-sense rules, and the pedestrian should do this no less than the driver. Our highway designs should include a shoulder amply wide to be used as a pathway, and so constructed as to be a serviceable path, firm, hard, and even. Pedestrians should walk on the left-hand side of the road and carry a lantern at night. These are details that must generally be developed by educational means. Regulation will do something to secure results, but fundamentally it is resolved into a matter of personal care and custom of the pedestrian.

EXAMINATIONS AND INSURANCE AS PROTECTION AGAINST IRRESPONSIBLE DRIVERS

Some form of examination of prospective drivers is required in a number of States. In some it is little more than a form. Certainly it should be searching in character, and carefully given. Many States require a permit to drive, but set no standard of proficiency and give no examination. Generally a minimum age below which a permit may not be received is fixed by the law. Certain advances can well be made in efforts to raise the standard of driving.

Every man on his own side. The Pan American Highway Commission on tour in North Carolina honors the white line

The examination system should be extended. The question of compulsory insurance has been advanced in at least one State, and if properly controlled is no doubt sound. It at least will protect the victim of accident from loss or damage caused by the reckless and irresponsible driver. It may also stiffen the examination system by coupling with it certain policy requirements of the insurance companies as to proficiency of drivers.

Certain penalties have also been considered that attempt "to make the punishment fit the crime" of violating the ethics of good driving. One is to deny to the driver the privilege of driving by revoking his license. In some cities and States this is now possible under the law. The law is not enforced as often as it should be, possibly because it is a punishment so entirely fitting that it seems to be too severe.

The safety of a single vehicle can be secured with comparative ease, economy, and some slight display of common sense by the driver. The matter is largely in his own hands. When the safety of passing vehicles is in question, however, there enter a number of new and important details, sometimes difficult of adequate solution. The much overworked word "pyschology" may be invoked because the simultaneous use of the highway by many drivers requires that each one shall have at least some assurance, some more or less definite idea of what each of the other is going to do in any ordinary situation arising. The simplest illustration of the requirement is the knowledge each driver has that one meeting him will turn to the right and pass on his left side. One of the important features of safety involves the extension of these simple practices to as many general conditions as possible.

To accomplish this result the education of the driver is necessary, and his observance of the instruction must be secured. Several States have devised simple schemes for impressing those in charge of motor vehicles with the elements of safe driving. Much has been written, there are certain "rules of the road," and safety will, in general, be promoted in proportion as these become developed, understood, and observed.

PRINCIPLES OF TRAFFIC REGULATIONS

Generally these are incorporated into ordinances called traffic regulations, and a great deal of study has been given to the proper methods of dealing with the peculiar problems involved. Traffic regulations should be devised to expedite the safe movement of traffic. As such they should cover in general the course of the vehicle on the road, its position when at rest, its speed, condition, control, and method of operation as affecting other vehicles. The rules should be simple and susceptible of rigid enforcement.

Obviously the advantage in traffic regulations is greatly increased by uniformity. The excellence of a regulation is increased by its uniform application elsewhere, because traffic is no longer only local in origin. The foreign car, so called, is common on most of our roads. They will become more so as road improvement is extended, and the use of identical regulations everywhere, on the rural road as well as in the city street, will make the highway safer. To this end the wider use of the same traffic and driving signals and the same signs as are now commonly used in cities should be effected so far as practicable.

Where marked differences now exist it is a good idea to see that the motorist coming from another State is informed of the salient features of the law and regulations by the use of a large, conspicuous sign at the State line, in which is set out clearly and briefly the important points of interest to the driver. Such signs are used by some States and a typical case is shown in the Connecticut sign on page 1.

Pennsylvania's cable guard rail is an effective safeguard

LAW ENFORCEMENT DIFFICULT BUT IMPERATIVE

It is clear that much can be accomplished by regulation and the driving habits formed by the enforcement of the rules in cities. The success of the regulation lies to a great extent in the way the law is enforced. In general, it is the common opinion that there is too much laxity. Recently there have been made again a number of suggestions, not new, but revised, for severe penalties. They are made almost annually or whenever a new sheaf of statistics of highway accidents is published. The principal suggestions are jail sentences

instead of fines, revocation of licenses, and impounding of the vehicle for a limited period. The National Automobile Chamber of Commerce has recently advocated impounding the car of the reckless driver.

The matter of strict enforcement of traffic laws is as difficult as it is imperative. The need of additional traffic police in cities has been apparent as traffic has increased until to-day the traffic squad is a large element in the personnel of the police department. To police the open highway in the same way is impracticable and unnecessary; but some form of more or less ubiquitous policing is essential and the creation of county and State police organizations seems to present a solution. Some States have already developed extremely efficient rural police corps. Massachusetts presents a conspicuous example of such a body of trained men. Maine, New York, Pennsylvania, and Maryland have rural or highway police and some States have county police. These men are provided with motor transportation and the rapidity with which they can patrol the roads, and the fact that they may appear anywhere at any time, gives them an efficiency and influence which is large in proportion to their number.

The law of North Carolina requires all vehicles to come to a complete stop at railroad crossings. The law is difficult to enforce and of questionable value

DIRECTION SIGNS AS AN ELEMENT IN SAFETY

A first aspect of the whole matter, and one that is possible of ready solution, is the protection of the traveling public against loss of direction and location. In cities this is effected by the common use of street names and signs. It should be systematized and extended to the entire highway system in such a way as to create a corresponding degree of certainty against the hazard of losing one's way. The long distance traveler is becoming more and more common and strangers to a locality should be able to count upon an intelligent system of directional signs, route markers, and other devices for indicating the direct and customary routes of the improved system of roads. This detail in the scheme of making the highways serviceable as well as safe has been highly developed in some States. Wisconsin has so carefully and fully marked the State road system that it has used the slogan, "It is harder to lose your way on Wisconsin highways than it is to find your way in some States." It should be made difficult to lose one's way anywhere on the entire national system of roads. To this end many States have already adopted distinctive markers which are now well known in many instances. In general these markers are most successful where they approximate the type used in Minnesota, whose star carrying the State route number is so generously scattered over the State. The keystone of Pennsylvania, the State outlines of South Dakota, Iowa, and Illinois, and the Indian head of North Dakota are samples of successful markers. Each of these is so designed that the sign is approximately 15 by 15 inches and each bears conspicuously the State route number. In all cases the design selected should be simple and should carry conspicuously the particular designation, usually a number.

The Michigan route marker

In this connection it may be noted that a system of numbering, properly devised and developed, offers one of the simplest and easiest schemes to apply to the designation of the large mileage and more or less complicated layout of a State-wide or nation-wide system of roads. So far only slight effort has been made to correlate the numbering in adjacent States and a large field for careful adjustment here presents itself. Minnesota and Iowa have continuously numbered routes and a few other States have aimed at a similar arrangement, but no concerted attempt has yet been made by all States.

UNIFORM NUMBERING SYSTEM UNDER CONSIDERATION

Such a plan is now being studied, however, by the Bureau of Public Roads and the Committee on Traffic Control and Safety of the American Association of State Highway Officials and it is hoped that a practicable arrangement will be worked out for presentation to the association at its next annual meeting in November. The plan under consideration is to adopt as numbers for the Federal-aid highway system, which is correlated

Whitewashed telephone poles, trees, and culvert headwalls to mark the road for night driving in Maryland

and connected throughout the entire country, all one and two digit numbers. This will give 50 odd and 49 even numbers and provide for 99 through routes. The odd numbers would be given to connecting and con-

(Continued on page 19)

PRACTICAL FIELD TESTS FOR SUBGRADE SOILS

SUGGESTED METHODS FOR MEASURING THE CHARACTER OF HIGHWAY SUBGRADES

By A. C. ROSE, Associate Highway Engineer, U. S. Bureau of Public Roads

THE character and condition of the subgrade, the kind and volume of the traffic, and the climatic conditions seem to be the three major factors which limit the life or usefulness of a road surface. The climatic conditions can be exactly determined from the records of the United States Weather Bureau. It is believed that an investigation of the trend and possibilities of economic development of a region together with traffic surveys should give a reasonably close estimate of the kind and volume of traffic. This article suggests a means of evaluating the third variable in the problem by outlining a method for measuring the character and condition of the subgrade.

It seems evident that it should be possible to prescribe the kind and dimensions of a road surface when the three factors outlined above have been evaluated for any given case and after existing satisfactory road surfaces (preferably local) have been observed. Although the increasing traffic is rapidly forcing a change in the design of road surfaces, it is believed that a survey of existing surfaces would mark a step forward in highway transport.

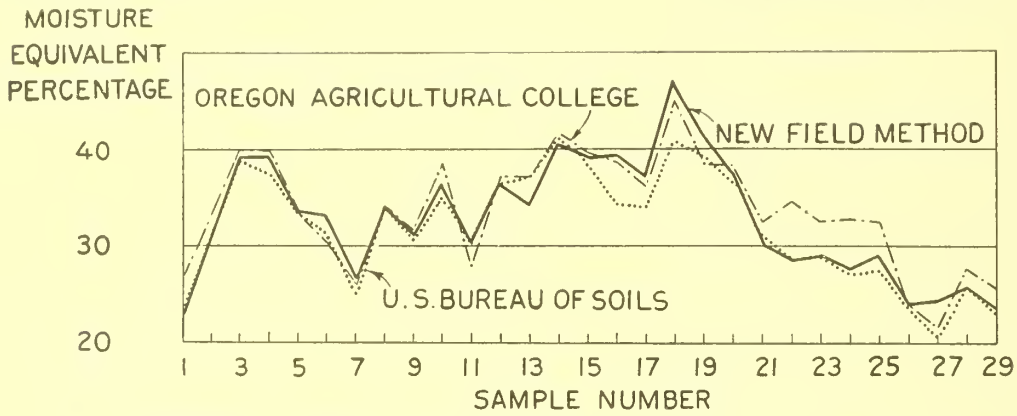


FIG. 1.—Comparison of field test for moisture equivalent with standard laboratory tests by the United States Bureau of Soils and Oregon Agricultural College

The methods herein outlined were evolved during a field study of road surfaces in the Pacific Northwest with a view to finding out the cause of certain road failures and developing methods of construction which would avoid a recurrence of similar conditions. The findings have been used to determine the character and dimensions of the surfacing on proposed road projects submitted for construction with Federal aid.

THE VARIABLE CHARACTER OF ROAD SUBGRADES

The character of road subgrades varies from solid rock through compacted gravel, hardpan, loose rock, gravel, sand, clay, colloidal clay, and swamp land to quicksand. Generally speaking, the bearing power of dry soils increases with the size of the particles. In the following table one recognizes immediately that the coarsest materials are those which possess the greatest supporting power.

	Diameter in millimeters
Fine gravel	2 to 1.
Coarse sand	1 to 0.5.
Medium sand	0.5 to 0.25.
Fine sand	0.25 to 0.10.
Very fine sand	0.10 to 0.05.
Silt	0.05 to 0.005.
Clay	less than 0.005.

Closely related to the size and character of the particles composing the soil is its ability to hold water against the force of gravity, i. e., its capillary or water-holding capacity, and this relationship is so definite, and the determination of the moisture-holding properties is so much simpler than mechanical analysis that it is preferred to the latter by the majority of investigators as a means of classifying soils.

The practical laboratory test is to determine the percentage of water the soil will hold in opposition to a force equal to 1,000 times the force of gravity, as developed in a centrifuge, and the percentage determined in this manner is known as the *moisture equivalent* of the soil.[1]

It has been found by experiment that the moisture equivalent percentage varies with the size and character of the soil grains. The smaller the grains, the greater is their surface area in a given volume of the soil and the larger is the moisture equivalent percentage. The character of the grain, whether glazed or rough, porous or solid, etc., also affects the test.

R. L. Pendleton states in regard to the moisture equivalent percentage that "This determination showed quite distinct averages for the types. Eliminating those samples shown to be nontypical according to the mechanical analysis, the variation within the type is reduced considerably. Yet it can not be said as regards this constant that all soils mapped under a given type name or even those soils under a given type name which the mechanical analysis has shown to be true to name have closely similar moisture equivalents. Briggs and McLane express the belief that ultimately moisture equivalent determinations will replace mechanical analysis in the classification of soils, because the determination is simple and the result can be expressed as a single constant."[2]

FIELD TEST FOR MOISTURE EQUIVALENT OF SOILS

Since the degree of fineness of the grains of a soil seems to determine its quality for road foundations, and since the moisture equivalent percentage has been found to bear a close relation to the degree of fineness it is desirable to develop some means of approximately determining the latter characteristic, without employing the centrifuge, the use of which is not practicable in the field. With this in mind the following field test has been devised:

The test is made by taking a 500-gram sample of air-dried soil, breaking up the lumps, placing the sample in a bowl, adding water slowly from a burette, mixing the water and soil until it reaches the consistency of

[1] For description of the standard laboratory test for moisture equivalent, see Public Roads, Vol. 4, No. 3, July, 1921, p. 16.

[2] Are soils mapped under a given type name by the Bureau of Soils method closely similar to one another? University of California Publications in Agricultural Science, Vol. 3, No. 12. Page 469.

putty and may be compacted with a spoon or spatula without any free water remaining on the surface. Water is then allowed to drop upon the smoothed surface as long as it is absorbed. Before the moisture equivalent percentage is reached the sample will absorb water readily but when the critical value is passed the surface will retain a wet, shiny appearance. The sample is then dried out at 105° C. and the percentage of water is calculated on the basis of the dry weight of the soil. This procedure eliminates the time and labor required to oven-dry and pulverize a sample. Pieces of rock larger than one-eight inch are removed.

TABLE 1.—*Results of moisture equivalent tests made by the United States Bureau of Soils*

Sample No.	Mechanical analysis—Per cent between Millimeters							Moisture equivalent percentage, standard method (centrifuge)
	2–1	1–0.5	0.5–0.25	0.25–0.1	0.1–0.05	0.05–0.005	0.005–0	
	Fine gravel	Coarse sand	Medium sand	Fine sand	Very fine sand	Silt	Clay	
1	0.0	0.0	0.4	1.5	12.3	64.9	20.9	23.8
2	0.0	0.6	1.0	6.4	21.2	46.5	24.4	30.7
3	0.0	1.8	3.0	11.6	16.4	39.3	28.1	38.9
4	0.0	1.3	1.5	7.7	8.5	39.6	41.4	37.5
5	0.0	2.9	2.7	9.0	12.9	41.0	31.3	33.8
6	0.2	0.5	0.5	4.5	10.9	49.3	34.0	31.3
7	0.0	0.3	0.4	4.1	10.9	59.6	24.6	25.2
8	0.0	1.0	0.8	5.4	14.8	46.0	31.9	33.9
19	0.0	0.3	0.5	4.3	10.7	47.2	37.0	30.5
10	0.0	0.9	1.2	8.0	8.2	39.1	42.6	34.9
11	0.0	0.0	1.1	3.3	13.0	48.3	34.3	30.5
12	0.0	1.6	1.7	8.2	9.4	40.7	38.3	36.4
13	0.0	1.8	1.8	10.7	11.3	35.2	39.3	37.5
14	0.0	2.5	2.2	8.5	6.7	34.6	45.4	41.2
15	0.0	7.2	5.8	19.4	12.7	28.2	26.7	38.5
16	0.0	1.5	1.6	8.4	10.5	36.7	41.3	34.4
17	0.0	0.6	0.9	6.1	10.0	46.3	36.2	34.1
18	0.0	1.3	1.8	8.7	10.6	37.6	40.0	40.8
29	0.0	3.9	3.2	26.3	28.8	25.8	11.9	39.1
20	0.0	0.4	0.6	6.5	22.8	39.7	30.0	36.3
21	0.0	0.7	0.9	10.7	24.9	35.7	27.1	30.9
22	0.0	0.6	0.5	3.0	17.8	53.4	24.6	28.3
23	0.0	0.4	0.3	2.7	22.8	47.6	26.2	28.7
24	0.0	0.4	0.5	3.0	21.3	47.2	27.6	27.2
25	0.0	0.4	0.3	3.0	12.0	60.3	24.1	27.5
26	0.3	0.3	0.5	2.9	22.3	61.1	12.2	23.2
27	0.1	0.5	0.8	4.8	22.6	61.4	9.2	20.4
28	0.0	0.2	0.3	2.0	23.5	50.9	23.0	25.7
29	0.0	1.4	2.0	4.9	19.3	56.8	15.6	23.0

TABLE 2.—*Moisture equivalent results by Oregon Agricultural College and field method*

Sample No.	Oregon Agricultural College		Difference A–B	United States Bureau of Public Roads, District 1		Difference C–D
	1st run A	2d run B		1st run C	2d run D	
1	26.95	27.02	0.07	22.5	22.9	0.4
2	34.03	33.98	0.05	31.1	30.2	0.9
3	40.12	39.98	0.14	38.5	40.0	1.5
4	40.50	39.34	1.16	39.1	39.5	0.4
5	34.16	34.01	0.15	33.7	33.7	0.0
6	30.68	30.47	0.21	33.3	32.6	0.7
7	25.87	26.01	0.14	26.2	26.8	0.6
8	34.16	33.94	0.22	34.2	34.0	0.2
9	31.45	31.58	0.13	31.8	30.7	1.1
10	38.01	38.64	0.63	35.9	37.0	1.1
11	28.12	27.94	0.18	30.1	30.4	0.3
12	37.44	37.03	0.41	36.6	36.2	0.4
13	37.49	37.42	0.07	34.2	34.2	0.0
14	41.54	42.09	0.55	39.8	41.4	1.6
15	39.78	39.42	0.36	38.8	39.6	0.8
16	38.58	38.90	0.32	39.4	39.2	0.2
17	36.19	36.23	0.04	37.2	37.2	0.0
18	45.09	45.27	0.18	47.3	46.8	0.5
19	38.45	38.79	0.34	41.7	41.7	0.0
20	38.73	38.21	0.52	37.3	37.2	0.1
21	32.49	32.51	0.02	30.2	30.0	0.2
22	34.31	34.89	0.58	28.2	28.6	0.4
23	32.73	32.22	0.51	28.2	29.3	1.1
24	32.96	32.48	0.48	27.4	27.7	0.3
25	32.32	32.53	0.21	27.2	30.6	3.4
26	23.54	23.50	0.04	23.2	24.5	1.3
27	21.13	21.68	0.55	24.1	24.4	0.3
28	27.95	27.34	0.61	26.3	24.8	1.5
29	25.31	25.42	0.11	23.4	23.6	0.2
Total differences			8.98			19.5

In order to determine how closely a refinement of this field method would agree with the results obtained by the standard laboratory test, 29 soil types varying from silt to clay were sampled and divided into three portions. One set of samples was oven-dried at 105° C. and pulverized to pass a 1-millimeter (1/25-inch) screen and then tested by the field method. The other samples were tested by the standard laboratory method by the United States Bureau of Soils and the Oregon Agricultural College, and the results of the three tests for the corresponding samples are shown in Figure 1. The appearance of a soil before, during, and after the field moisture equivalent test has been made is shown in Figures 2, 3, and 4.

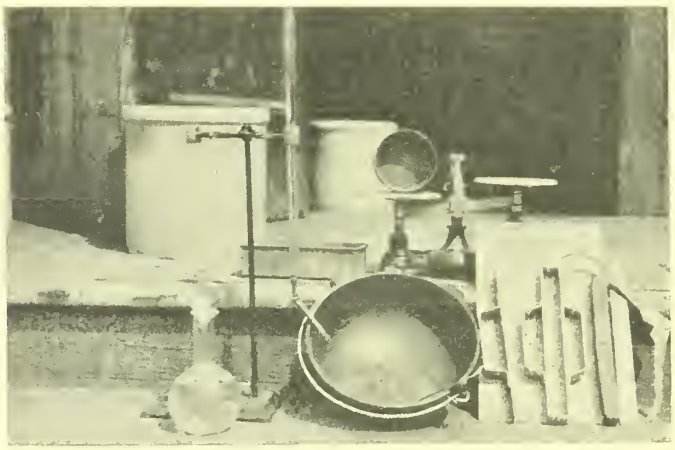


FIG. 2.—Oven-dried soil pulverized to pass a 1-millimeter ($\frac{1}{25}$-inch) screen

FIG. 3.—Same soil wetted to approximately one-half the moisture equivalent percentage

FIG. 4.—Same soil wetted to the moisture equivalent percentage

The mechanical analysis and the moisture equivalent results of the United States Bureau of Soils are given in Table 1. The difference between the original and check tests by the Oregon Agricultural College and the field method are shown in Table 2. The sum of the differences is decidedly in favor of the laboratory method although the results seem to indicate that the field method is close enough for all practical purposes.

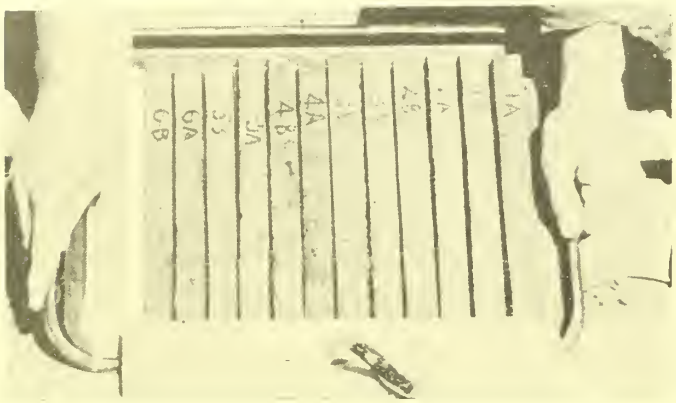


FIG. 5.—Soil bars, wetted to the moisture equivalent percentage, before drying

The moisture equivalent results by the three agencies and the residuals are given in Table 3. Considering the results of the Bureau of Soils as correct, the probable errors, by the method of least squares, for the Oregon Agricultural College and the field method are ± 1.9 and ± 1.4 respectively.

FIG. 6.—Same soil bars after being oven-dried at 105° C.

FIELD METHOD FOR DETERMINING THE LINEAL SHRINKAGE PERCENTAGE OF SOILS

It is desirable, also, to determine the lineal shrinkage percentage of various soil types because the vertical displacement or distortion of the subgrade, resulting from the shrinkage or swelling of the soil, is a factor which limits the life of a road surface, and what appears to be a practical field method of making this test has also been developed. It is described as follows:

Procedure.—A 300-gram sample of soil wetted to the moisture equivalent percentage is packed in ½-inch layers in a galvanized iron mold with a wooden tamper (½ inch by 1½ inches by 14 inches). The bars after being weighed are pushed from the mold upon a porcelain plate and calipered and then oven-dried at 105° C. and calipered again. (See figs. 4, 5, and 6.) The difference in length computed as a percentage of the wet length of the bar is considered the lineal shrinkage percentage. The volumetric shrinkage percentage is computed on the assumption that the shrinkage is uniform in all three dimensions. A computed graph showing the relation between the lineal and volumetric shrinkage factor is given in Figure 7.

Original and check runs made to determine the lineal shrinkage percentage of the 29 soil samples previously described are shown in Table 4, from which it will be

TABLE 3.—*Moisture equivalent percentages as determined by the three agencies*

Sample No.	Moisture equivalent percentages			Differences		Residuals	
	United States Bureau of Soils (A)	Oregon Agricultural College (B)	United States Bureau Public Roads District 1 (C)	A–B	A–C	$(A-B)^2$	$(A-C)^2$
1	23.8	27.0	22.7	−3.2	+1.1	10.24	1.21
2	30.7	34.0	30.6	−3.3	+0.1	10.89	0.01
3	38.9	40.0	39.2	−1.1	−0.3	1.21	0.09
4	37.5	39.9	39.3	−2.4	−1.8	5.76	3.24
5	33.8	34.1	33.7	−0.3	+0.1	0.09	0.01
6	31.3	30.6	33.0	+0.7	−1.7	0.49	2.89
7	25.2	25.9	26.5	−0.7	−1.3	0.49	1.69
8	33.9	34.0	34.1	−0.1	−0.2	0.01	0.04
9	30.5	31.5	31.2	−1.0	−0.7	1.00	0.49
10	34.9	38.3	36.4	−3.4	−1.5	11.56	2.25
11	30.5	28.0	30.2	+2.5	+0.3	6.25	0.09
12	36.4	37.2	36.4	−0.8	0.0	0.64	0.0
13	37.5	37.4	34.2	+0.1	+3.3	0.01	10.89
14	41.2	41.8	40.6	−0.6	+0.6	0.36	0.36
15	38.5	39.6	39.2	−1.1	−0.7	1.21	0.49
16	34.4	38.7	39.3	−4.3	−4.9	18.49	24.01
17	34.1	36.2	37.2	−2.1	−3.1	4.41	9.61
18	40.8	45.2	47.0	−4.4	−6.2	19.36	38.44
19	39.1	38.6	41.7	+0.5	−2.6	0.25	6.76
20	36.3	38.5	37.2	−2.2	−0.9	4.84	0.81
21	30.9	32.5	30.1	−1.6	+0.8	2.56	0.64
22	28.3	34.6	28.4	−6.3	−0.1	39.69	0.01
23	28.7	32.5	28.8	−3.8	−0.1	14.44	0.01
24	27.2	32.7	27.6	−5.5	−0.4	30.25	0.16
25	27.5	32.4	28.8	−4.9	−1.3	24.01	1.69
26	23.2	23.5	23.8	−0.3	−0.6	0.09	0.36
27	20.4	21.4	24.2	−1.0	−3.8	1.00	14.44
28	25.7	27.6	25.6	−1.9	+0.1	3.61	0.01
29	23.0	25.4	23.5	−2.4	−0.5	5.76	0.25
Totals						218.97	120.95

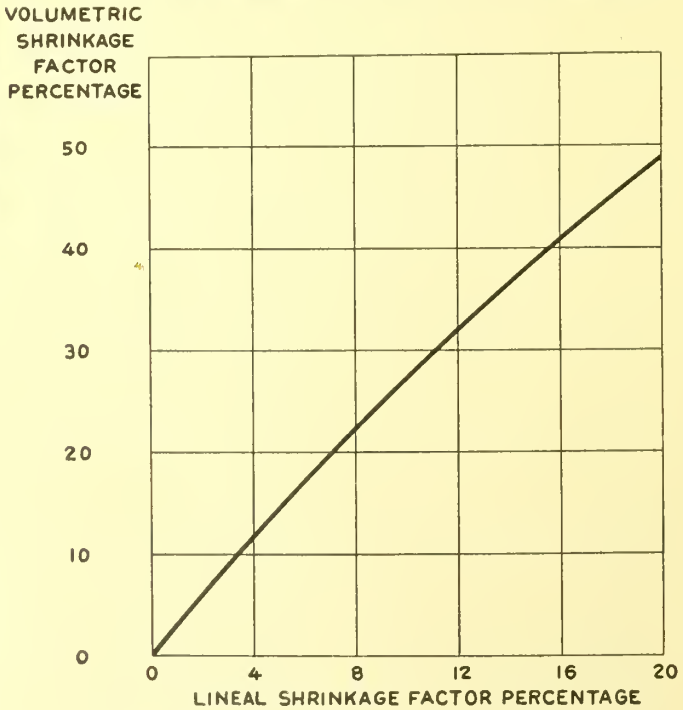


FIG. 7.—Relation between lineal and volumetric shrinkage

noted that the two determinations check very closely, and the relation of lineal shrinkage to the moisture equivalent of the same samples is represented by Figure 8.

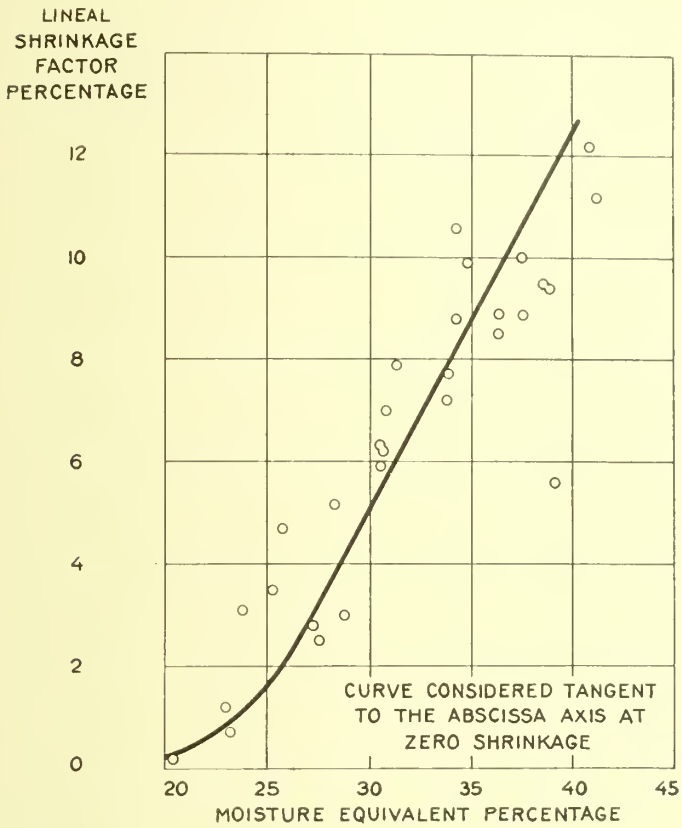


FIG. 8.—Relation of lineal shrinkage to moisture equivalent for 29 soil samples from Oregon Federal-aid project No. 51

TABLE 4.—*Field determinations of lineal shrinkage of soils*

Sample No.	Length of compacted bar in inches				Shrinkage of bar					Volumetric shrinkage (computed)	Moisture equivalent
	Wet		Dry		1st run	2d run	1st run	2d run	Average observed lineal shrinkage		
	1st run	2d run	1st run	2d run							
					Inches	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	10 8/64	10 2/64	9 52/64	9 47/64	20/64	19/64	3.1	3.0	3.05	8.8	22.7
2	10 4/64	10 3/64	9 27/64	9 29/64	41/64	38/64	6.4	5.9	6.15	17.25	30.65
3	10 2/64	10 2/64	9 6/64	9 3/64	60/64	63/64	9.3	9.8	9.55	26.2	39.25
4	10 2/64	10 1/64	9 2/64	9 1/64	64/64	64/64	10.0	10.0	10.0	27.1	39.30
5	10 4/64	10 0/64	9 2/64	9 19/64	66/64	45/64	10.2	7.0	8.6	23.6	33.70
6	10 2/64	10 4/64	9 15/64	9 17/64	51/64	51/64	7.9	7.9	7.9	21.75	32.95
7	10 2/64	10 0/64	9 44/64	9 41/64	22/64	23/64	3.4	3.6	3.5	10.2	26.50
8	10 2/64	10 2/64	9 16/64	9 17/64	50/64	49/64	7 8	7.6	7.7	21.5	34.10
9	9 63/64	10 2/64	9 21/64	9 27/64	42/64	39/64	6.6	6.1	6.35	17.8	31.25
10	10 2/64	10 1/64	9 4/64	9 1/64	62/64	64/64	9.7	10.0	9.85	26.5	36.45
11	10 1/64	10 0/64	9 26/64	9 28/64	39/64	36/64	6.1	5.6	5.85	16.5	30.25
12	10 1/64	10 2/64	9 9/64	9 8/64	56/64	58/64	8.7	9.0	8.85	24.25	36.40
13	10 2/64	10 1/64	9 8/64	9 9/64	58/64	56/64	9.0	8.7	8.85	24.2	34.20
14	10 1/64	10 2/64	8 57/64	8 58/64	72/64	72/64	11.2	11.2	11.2	30.0	40.60
15	10 2/64	10 2/64	9 5/64	9 5/64	61/64	61/64	9.5	9.5	9.5	25.9	39.20
16	10 2/64	10 2/64	8 63/64	8 61/64	67/64	69/64	10.4	10.7	10.55	28.3	39.30
17	10 2/64	10 3/64	9 9/64	9 11/64	57/64	56/64	8.9	8.7	8.8	24.25	37.20
18	10 3/64	10 2/64	8 52/64	8 52/64	79/64	78/64	12.3	12.1	12.2	32.2	47.05
19	10 2/64	10 3/64	9 31/64	9 31/64	35/64	36/64	5.5	5.6	5.55	15.8	41.7
20	10 2/64	10 1/64	9 11/64	9 11/64	55/64	54/64	8.6	8.4	8.5	23.5	37.25
21	10 2/64	10 0/64	9 21/64	9 19/64	45/64	45/64	7.0	7.0	7.0	19.6	30.10
22	10 2/64	10 0/64	9 31/64	9 30/64	35/64	34/64	5.5	5.3	5.4	15.3	28.40
23	10 0/64	9 63/64	9 45/64	9 44/64	19/64	19/64	3.0	3.0	3.0	8.6	28.75
24	10 0/64	9 63/64	9 46/64	9 45/64	18/64	18/64	2.8	2.8	2.8	8.3	27.55
25	9 62/64	9 62/64	9 47/64	9 46/64	15/64	16/64	2.3	2.5	2.4	7.0	27.90
26	9 60/64	9 62/64	9 57/64	9 57/64	3/64	5/64	0.5	0.8	0.65	2.0	23.85
27	9 62/64	9 58/64	9 62/64	9 56/64	0/64	2/64	0	0.3	0.15	0.4	24.25
28	10 1/64	10 2/64	9 33/64	9 38/64	32/64	28/64	5.0	4.4	4.7	14.0	25.55
29	9 58/64	9 63/64	9 52/64	9 54/64	6/64	9/64	0.9	1.4	1.15	3.4	23.50

A ROUGH PRACTICAL METHOD OF CLASSIFYING SUBGRADE SOILS

Using these field tests a series of studies of road subgrades have been made in the States of Oregon and Washington which are included in Bureau of Public Roads District No. 1, as one of the results of which a method of roughly classifying subgrade soils by means of a soil classification triangle, shown in Figure 9, has been evolved. This triangle is plotted according to Professor Ferêt's method (Annales Des Ponts et Chaussées, 1892). Its use is based on the fact that in any equilateral triangle, the sum of the perpendiculars from any point to the three sides is equal to the altitude of the triangle. Thus at intersection A, the soil contains 37.6 per cent silt, 6.7 per cent sand, and 55.7 per cent clay, and is classified CLAY. The soil nomenclatures shown by the shaded areas are believed to be representative of the practice of the leading authorities. The percentages represent the results obtained by a mechanical analysis. The horizontal lines represent the percentages of clay which, it is believed generally, determine the quality of the soil for a road subgrade. Thus it is believed that, as a rule, a soil containing less than 20 per cent of clay makes a good subgrade, one containing from 20 to 30

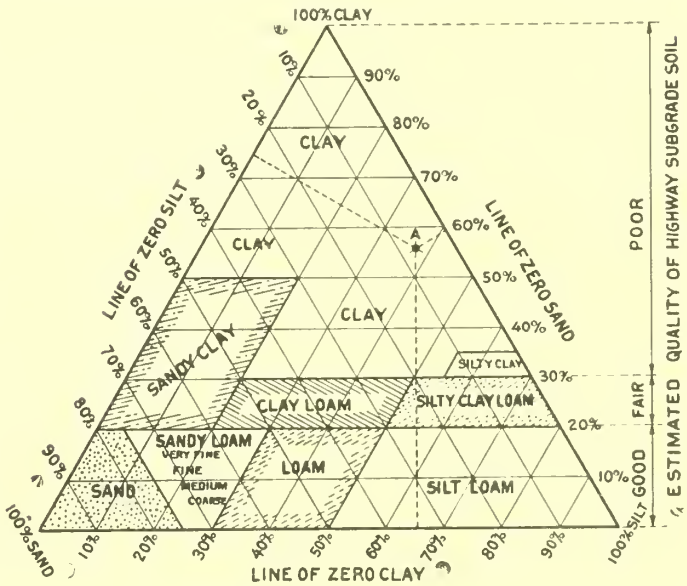


FIG. 9.—Trilinear soil classification chart

per cent of clay a fair subgrade, and one containing more than 30 per cent a poor subgrade.

The field observations of the bureau indicate that, for road soil reconnaissance survey purposes, the clay content by mechanical analysis may be considered as equal to the moisture equivalent percentage. Adopting this assumption (see Table 1) the basis of the above classification is apparent in Figure 8. As shown by this figure the field observations indicate that when the moisture equivalent is less than 20, the lineal shrinkage is too small to be measured and the soil makes a good subgrade; when the moisture equivalent percentage varies from 20 to 30 the average maximum lineal shrinkage is about 5 per cent and the soil makes a fair subgrade; but when the moisture equivalent exceeds 30 per cent, the average maximum lineal shrinkage becomes excessive and the soil makes a poor subgrade. Therefore, in regions where soil survey

bulletins are available, this chart may be used in making a reconnaissance survey of the subgrade soil on a road project. But these results should be verified in all cases by actual tests, at least until the soil types of the region have been evaluated.

OTHER FINDINGS OF THE SUBGRADE STUDIES

Other major findings of the bureau's subgrade studies follow:

1. The field moisture equivalent test seems to give results practically identical with the standard laboratory methods.

2. A moisture equivalent percentage of 20 seems to be critical in respect to the bearing power of a soil. When the moisture equivalent percentage is less than

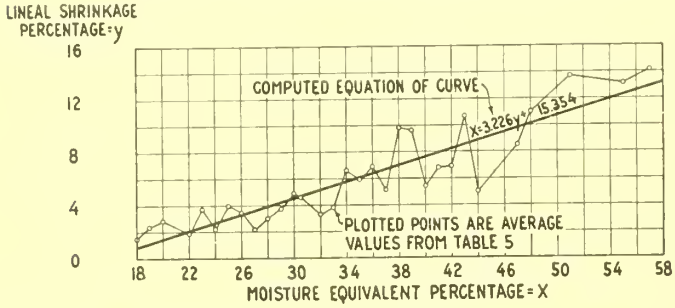


FIG. 10.—Lineal shrinkage percentage curve computed from results obtained from tests of 176 subgrade soils in Oregon and Washington

20 the density and possibly the bearing power of a soil do not seem to be decreased perceptibly by the addition of water until the moisture content is made greater than the total voids in the soil.

3. Defining the term "stability ratio" as the actual moisture content percentage of the soil divided by its moisture equivalent value, the field investigations in the States of Oregon and Washington seem to indicate that the bearing power of a soil is relatively low when the stability ratio is greater than unity. When the stability ratio is less than unity, existing subgrades are generally well compacted, firm and hard. The application of the stability ratio is limited to soils with moisture equivalent percentages greater than 20.

4. When the moisture equivalent percentage of a soil is greater than 20 the moisture equivalent percentage seems to represent the extreme limit that the moisture content of a subgrade should be permitted to reach.

5. The lineal shrinkage percentage test detects poor subgrade soils which pass a favorable moisture equivalent test or vice versa.[3]

6. The lineal shrinkage of a soil with a moisture equivalent percentage of less than 20 is usually too small to be measured. According to the curve in Figure 10 the shrinkage will be zero when the moisture equivalent percentage is 15.4 per cent. This curve was computed from the results of 176 tests. An original and check run was made for each test and the average result used. For all practical purposes it is believed that zero shrinkage may generally be assumed to occur when the moisture equivalent percentage is equal to or less than 20. Table 5 gives the maximum, minimum and average values of the lineal shrinkage percentages for the corresponding moisture equivalent percentages expressed to the nearest unit figure.

7. When the lineal shrinkage percentage is less than 5 per cent, the subgrade soil does not seem to require special treatment or protective measures to prevent the shrinkage or swell from reducing the life of the road surfacing.

[3] A soil with porous grains might have a high moisture equivalent and a relatively low shrinkage percentage. Soils with a high shrinkage percentage contain larger percentages of clay and heave readily when frozen. In the State of Washington 81 per cent of the cement concrete pavement slabs are cracked in the eastern portion where the frost is severe, while in the western third of the State where the frost is severe only 55 per cent of the slabs subjected to heavy logging traffic are cracked and 15 per cent of those not subjected to heavy logging traffic (1922 data.)

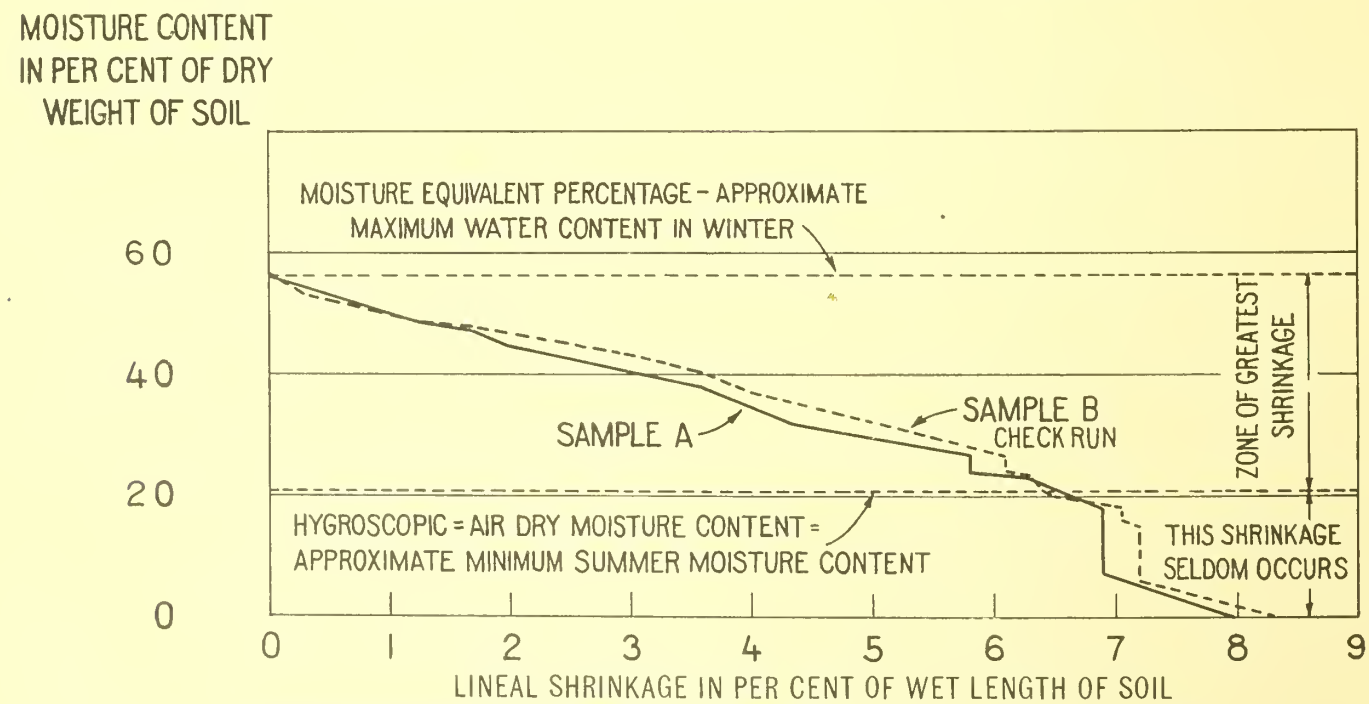


FIG. 11.—Relation of lineal shrinkage to moisture content on sample of clay subsoil from Pacific Highway near Woodland, Wash.

8. When the lineal shrinkage percentage is greater than 5 per cent,[4] it seems that special precautions should be taken to insure a road surface against failure due to subgrade soil movement. The moisture content of the soil when in excess of capillary moisture may be reduced by surface or tile drains or both. The pavement may be strengthened by increased depth, steel reinforcement, or a suitable subgrade. A sand cushion may be provided under the surface or the soil may be treated. It is to be noted that the actual variation in the moisture content of the soil determines the amount of shrinkage or swelling. Only in rare instances do soils dry out completely and reach the maximum possible shrinkage percentage. Figure 11 illustrates this.

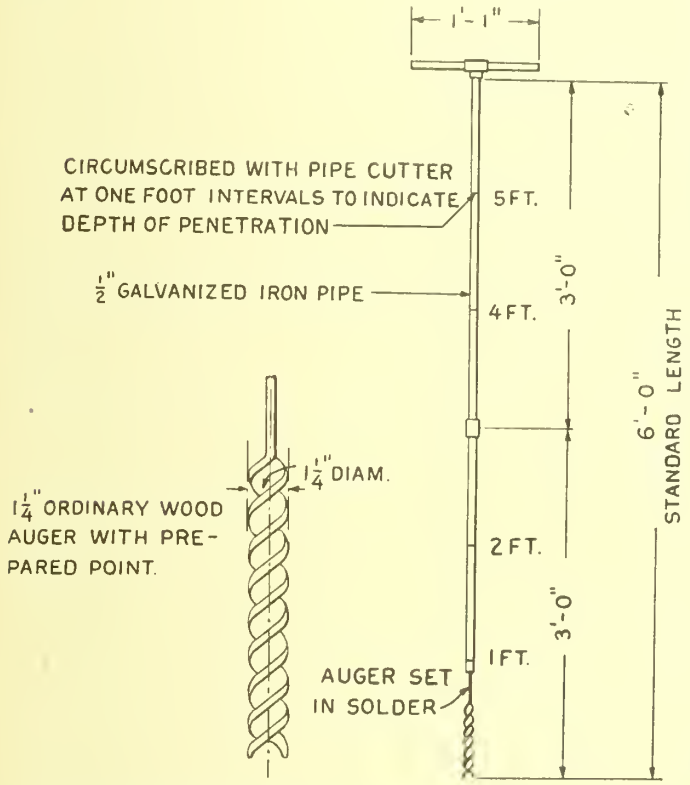


FIG. 12.—Soil auger. Extra pipe lengtbs may be added in 3-foot sections. The total practicable penetration is 18 feet. The point is prepared as follows: (1) Anneal; (2) saw or drill out concave center; (3) trim witb round file, without bending or altering conformation of prongs; (4) sharpen ends of two prongs to chisel points, ¼-inch wide, with flat surface in plane perpendicular to axis of drill

9. Embankments across clay soil lowlands which are subject to flood should, wherever possible, be constructed with soil hauled from the surrounding hills rather than by side borrow. The lowland clay generally has a high lineal shrinkage percentage, while the upland soil has usually a much lower shrinkage value. The core of the fill may be made by side borrow and sloped with upland material, if the cost of the long haul prohibits the formation of the entire embankment from the upland soil.[5]

TABLE 5.—*Lineal shrinkage percentages based on tests of 176 soils in Oregon and Washington*

Field moisture equivalent to nearest unit	Lineal shrinkage			Number of tests	Field moisture equivalent to nearest unit	Lineal shrinkage			Number of tests
	Maximum	Minimum	Average			Maximum	Minimum	Average	
Per cent	*Per cent*	*Per cent*	*Per cent*		*Per cent*	*Per cent*	*Per cent*	*Per cent*	
18	3.0	0.0	1.5	2	38	12.5	6.4	9.8	4
19	2.4	2.4	2.4	1	39	10.6	8.3	9.6	5
20	3.4	2.4	2.9	2	40	9.4	1.9	5.4	4
21				0	41	11.2	1.0	6.8	6
22	3.1	1.1	1.9	6	42	11.3	3.7	6.9	5
23	6.7	2.5	3.7	6	43	11.2	10.3	10.7	3
24	5.3	.2	2.1	9	44	6.4	3.6	5.0	2
25	6.5	1.7	4.0	9	45				
26	10.9		3.5	14	46				
27	4.7	.5	2.1	6	47	12.2	5.6	8.5	3
28	5.2	1.6	3.0	11	48				
29	7.5	2.6	3.7	5	49	11.8	10.2	11.0	2
30	7.0	2.8	4.9	11	50				
31	9.0	1.0	4.6	16	51	13.7			1
32	4.9	2.1	3.3	8	52				
33	7.9	.6	3.8	5	53				
34	12.9	1.3	6.6	10	54				
35	8.2	2.1	5.9	6	55	13.1			1
36	9.9	3.5	6.9	8	56				
37	8.8	1.0	5.1	4	57	14.1			1

EQUIPMENT REQUIRED FOR A ROAD PROJECT SOIL SURVEY

The apparatus and equipment required to make subgrade soil surveys similar to these described is as follows:

1 automobile (preferably a closed car, since the best investigations of drainage are made during rainy weather).
1 Soil auger (see fig. 12).
6 dozen galvanized seamless salve boxes 3 inches in diameter and 2½ inches deep with covers.
1 small camp oven, preferably equipped with a thermostat.
1 electric hot plate for heating oven.
1 small pick.
1 shovel.
1 set of equipment for moisture equivalent and shrinkage tests as shown in Figure 2.

After the moisture equivalent and shrinkage values of typical soils in a region have been determined, it is necessary to make routine tests only in rare instances. An experienced investigator is able to identify silts, loams, clays, sands, etc., by rubbing between the fingers and it is remarkable how closely the moisture equivalents may be estimated by inspection.

[4] This is equivalent to a volumetric shrinkage factor of about 14 per cent and corresponds approximately to a moisture equivalent percentage of 30.

[5] One of the most serious subgrade failures in the Pacific northwest occurred on the Pacific Highway between Centralia and Chehalis, Wash. The fill had been made by side borrow with the heavy Chehalis clay soil which had a moisture equivalent of 55 per cent and a lineal shrinkage percentage of 13 (equivalent to a volumetric shrinkage percentage of 34). The berms left at the toe of the slope of the fills were too narrow. The embankment was distorted when the soil was swelled annually by flood waters from the Chehalis River. The fill sloughed into the borrow pits and left the cement concrete pavement unsupported under the edges in many places. The impact and weight of the traffic resulted in many longitudinal and transverse cracks and broken corners. The character of the concrete pavement was good. This was a typical subgrade failure. To remedy this failure the State highway department has widened the old clay fill by placing a layer of upland soil from 4 to 8 feet in thickness along the side slopes. This new fill soil has a comparatively low shrinkage percentage. After ultimate settlement of the new embankment, the existing pavement will be resurfaced with a cement concrete pavement 20 feet wide. Precaution will be taken to provide a sliding plane between the old and new pavements.

THE REGULATION OF MOTOR VEHICLES AS COMMON CARRIERS

By HENRY R. TRUMBOWER, Economist, U. S. Bureau of Public Roads

IN 1907, about the time when automobiles began to come into general use, the States were still debating whether public service corporations furnishing light, heat, power, water, and telephone service and the traction lines should be placed under the regulation of the States by public utility commissions. By that time most of the States had established railroad commissions which regulated the rates and service of steam railroads and there was a general feeling that these other public utility enterprises should be similarly regulated both as to rates and service. Wisconsin and New York were the first States to extend this jurisdiction of the State and both enacted laws which gave these regulatory powers to railroad commissions which already existed.

When the automobile began to be used as a common carrier, either as a jitney competing with city street-car service or as a bus between cities or as a truck carrying persons and property for compensation, it was recognized that this was a new type of public service. Cities first began regulating these new transportation agencies and subsequently State legislatures began passing laws which extended their jurisdiction and regulatory power to the motor vehicle operated as a common carrier in like manner as to the rail transportation service. There has been considerable discussion as to the policy of extending a State's regulatory power to the motor vehicle. As yet there is no complete agreement on the subject and not all the States have gone so far as to regulate common carrier motor vehicles to the same degree as steam and electric railroads.

The principal arguments in favor of State regulation of automobile carriers may be summarized as follows: That motor vehicles operating as common carriers should be regulated the same as other public transportation agencies and in that manner travelers and the shipping public could be assured regular, adequate, and efficient service including at the same time the highest degree of safety and responsibility; that where motor vehicle common carriers are apt to bring about competitive conditions which are ruinous to other carriers, the State can determine what is to the best interest of the public in the long run and protect that agency which it is shown deserves protection and whose service is needed by the public; that there should be placed upon the common carrier motor vehicle those duties and obligations which are commensurate with the benefits obtained from the public which furnishes the highway and allows it to be used for gain and profit; that public regulation is desirable in order to protect from irresponsible competitors those motor vehicle common carriers which have established themselves and are furnishing satisfactory service.

IS REGULATION NECESSARY IN ABSENCE OF MONOPOLY?

The attitude of those opposing this type of legislation finds its expression in the message of the governor accompanying the veto of a motor-vehicle regulation measure passed by the Wisconsin Legislature. He said: "My objection to this bill is fundamental. Jitneys and busses may be operated by anyone upon the public highways and streets, and therefore there is no opportunity to create a monopoly. Free competition prevails, and thus rates and services are regulated by the natural law of competition. It is quite different with respect to a street railway, an interurban railway, or a railroad. When either of those occupies a territory, there is no opportunity for a competitive, like system to come into that territory. There is a limitation on the number of street railways or interurban lines that may occupy the field of transportation, and so the transportation companies hold the field against all others; and the reason for regulating them is because of the fact that they possess a monopoly, affording them the right, in the absence of regulation, to arbitrarily discriminate, grant rebates and other special favors, charge an excessive fare, and give inadequate service. The legislature, therefore, wisely provided for the regulation of such monopolies. The motor vehicles have come to stay; they are the beginning of a transportation system about which it is dangerous to prophesy. They may in the future be displaced by more modern systems of transportation. The old canal companies fought for their existence when the railroads came, but the building of railroads could not be obstructed merely for the purpose of preserving the canal companies. * * * I do not believe that it is any part of the State's duty to obstruct or hinder that means of conveyance. So long as the motor vehicles do not possess a monopoly, there is no ground for regulating them, except to protect the safety and health of the traveling public. To regulate them from the standpoint of protecting some other business, is to interfere with free competition, not in the interests of the general public, but for the special protection of some specific business."

There is a certain amount of truth in the statement that public regulation is not necessary where monopoly conditions do not prevail. Regulation on the part of the Federal Government and the State governments was undertaken for the purpose of protecting the public against unreasonable rates and discriminatory practices which grew up under monopoly conditions. It seems, therefore, logical that when and if competitive conditions are restored regulation can be given up. In this connection it should, however, be observed that the motor vehicle as a common carrier makes its appearance as a competitor in the entire field of transportation, including steam railroads and electric city and interurban railroads. To keep on regulating one form of transportation as to rates and service and allow the new type of competitor free rein without placing upon him any degree of restraint or responsibility is an unfair practice and will lead sooner or later to conditions which will be intolerable from the public point of view. The public is entitled to adequate and continuous service at reasonable rates; unlimited competition on the part of the motor vehicle with the common carrier which is regulated by the public will in the long run seriously affect the service and may force that type of carrier to curtail or abandon its service which is most essential to public needs.

MOTOR VEHICLE COMMON CARRIERS REGULATED IN 28 STATES

At the end of 1923 there were 28 States[1] which had adopted the definite policy of regulating motor vehicles engaged in common carrier service and through the State public service commissions were exercising control over the routes, service, and fares. Many of the cities exercise control over so-called jitney operation even though the State has not enacted any legislation pertaining to motor vehicle common carrier service on the public highways. There are also certain States which have not gone so far as to put automobiles carrying passengers and freight for hire into the public utility class which at the same time require such operators to carry liability insurance and give bonds. As yet the Federal Government has not taken any steps to regulate automobile carriers although there are a great many who engage in interstate business. Where railroad companies have made arrangements to forward freight shipments by rail and motor truck the Interstate Commerce Commission has taken the position that the commission's jurisdiction does not extend to that part of the service carried on by motor truck over the highways.

The right of the State to regulate the activities of motor vehicles engaged in common carrier operation is well established; this is true whether the State does it directly through a regulating commission or through a city council which derives its powers from a charter granted by the State. The regulation and control of the jitney has been held by the courts to be a legitimate exercise of the police power. *Memphis* v. *State ex rel. Ryals.*, 133 Tenn. 83; *Ex parte Dickey*, 76 W. Va. 576; *Auto Transit Co.* v. *Fort Worth (Texas)*, 182 S. W. 685; *Thielke* v. *Albee*, 79 Ore. 48; *Huston* v. *Des Moines*, 176 Iowa 455; *Green* v. *San Antonio (Texas)*, 178 S. W. 6; *Smith* v. *Nunnelly (W. Va.) P. U. R.*, 1915E 177. Under the police power of a State the legislature may prescribe the number, character, routes, rates, and hours of service of common carrier vehicles on highways; this power can be delegated to a city council or to a public service commission.

In framing laws of this character it is necessary in the first place to name and describe definitely the type of motor vehicle and the nature of the service which is to be regulated and supervised by the State. The State of Washington has adopted a very comprehensive definition covering this matter; its jurisdiction extends over all "auto transportation companies" and these are defined as "every corporation or person, * * * owning, controlling, operating, or managing any motor propelled vehicle not usually operated on or over rails used in the business of transporting persons, and, or, property for compensation over any public highway in this State between fixed termini or over a regular route, and not operated exclusively within the incorporated limits of any city or town; Provided, that the term auto transportation company, as used in this act, shall not include corporations or persons * * * in so far as they own, control, operate, or manage taxicabs, hotel busses, school busses, motor propelled vehicles, operated exclusively in transporting agricultural, horticultural, or dairy or other form of products from the point of production to the market." [2]

One of the frequent questions which commissions and courts have to decide is whether or not a specific motor vehicle is operated in such manner so as to bring it within the State's regulatory powers. Statutory definitions which are clear and comprehensive are of great assistance to those who are responsible for the administration of the law. That the State has the power to regulate common carriers is no longer open to argument, but a commission can regulate only those common carriers which have been definitely named and included by definition in the statute conferring jurisdiction to the regulatory agency.

An example of this came up in California. The California commission held that a motor bus line, auto truck line, or auto stage line engaged in transportation for hire was not a public utility under the California act although it might be a common carrier. *Western Assoc.* v. *Hackett (Calif.)* P. U. R., 1915F. 997. The position taken by the commission was reversed by the State supreme court when the case came before it on appeal; the court held that such motor vehicle lines were "transportation companies" and subject to the control of the commission because the constitution from which the commission received its authority gave the commission the power to regulate railroads and "other transportation companies." *Western Assoc.* v. *Railroad Commission*, 173 Calif. 802. In following the rules of construction the court must have concluded that motor vehicle common carriers resembled "railroads" closely enough so as to come within the category of "other transportation companies." In a great number of States which have had public utility acts and where power was given to commissions to regulate railroads, both steam and electric, the laws have been amended by adding motor vehicles engaged in common carrier service and by putting them in the same class as railroads. The Colorado Public Utilities act defines the term "common carrier" in saying: "The term 'common carrier' when used in this act includes every railroad, corporation * * * and every other corporation or person affording a means of transportation by automobile or other vehicle whatever, similar to that ordinarily afforded by railroads or street railways and in competition therewith." [3]

The statute of Maryland, which was one of the first States to regulate the motor vehicle as a common carrier, provided that the term "common carrier" includes all persons * * * operating automobiles and motor cars or motor vehicles for public use, in the conveyance of persons or property.[4] The completeness of the jurisdiction which the States have because of statutory enactments depends wholly upon the specific provisions of the law in each case. In most States the powers granted to the commissions are far reaching in character; in some States the jurisdiction applies to passenger motor vehicles alone; in New Jersey the commission has jurisdiction over and can regulate only those auto busses which operate over routes parallel to a street railway line or traction railway.[5] The Ohio law limits the commission to a supervision of the rates and service and safety of operation of common carrier motor vehicles; the control of competition with other transportation agencies is not delegated to the commission.[6]

[1] Arizona, California, Colorado, Connecticut, Georgia, Illinois, Iowa, Maine, Maryland, Michigan, Montana, Nevada, New Hampshire, New Jersey, New York, North Dakota, Ohio, Oklahoma, Oregon, Pennsylvania, Rhode Island, South Dakota, Utah, Vermont, Virginia, Washington, West Virginia, Wyoming.

[2] Chapter 111, sec. 1, Laws of Washington, 1921.

[3] Chap. 127, sec 2-E, Laws of Colorado, 1915.

[4] Chap. 180, sec. 1. 5, Laws of Maryland, enacted 1910, amended by Chap. 415, laws of 1914.

[5] Chap. 149, sec. 15, laws of New Jersey, 1921.

[6] Act July 26, 1913.

CERTIFICATES OF PUBLIC CONVENIENCE AND NECESSITY

Wherever State commissions exercise complete control and supervision of common carrier motor vehicle transportation it is necessary that any one desiring to engage in this kind of service obtain from the commission a permit which certifies that the holder has the right to give service of a certain character between two definite terminals. This permit or certificate is not issued unless it is shown that public convenience and necessity require this new service or operation and consequently the certificates are generally called "certificates of public convenience and necessity." This procedure is taken directly from the railroad and public-utility laws and is the basis of the monopoly feature of such enactments. The New York Public Service Commission in one of its decisions has made a very good attempt at defining this term. "Taking the phrase as an entity," the commission says, "it does not mean to require a physical necessity or an indispensable thing. We take it that for such purposes as are involved in this and similar applications, a public convenience and necessity exists when the proposed facility will meet a reasonable want of the public and supply a need, if existing facilities, while in a sense sufficient, do not adequately supply their need." [7] In most States where laws have been passed requiring that no motor vehicles operate as common carriers until they have secured public convenience and necessity certificates it is provided that those operating at the time the law becomes effective are considered as being within their rights. The certificates may be good only for the year issued or they may be granted for an indefinite period; if granted for a year they are ordinarily renewed at the end of the year without any further formalities.

A good deal of the litigation before commissions and courts involving the regulation of the operations of motor vehicles as common carriers pertains to contests over the granting of a certificate of public convenience and necessity. Practically all of these cases grow out of applications to operate motor bus lines. The cases can be grouped into two classes; *first*, where an applicant seeks the right to operate in a field which is already occupied; *second*, where there are several applicants seeking the right to operate in an unoccupied area. The competition which would be set up in this first group of cases is with street railways in cities or in suburban sections, with steam railroads, and with other common carrier motor vehicles.

STREET RAILWAYS ENTITLED TO PROTECTION AGAINST LOSS OF SHORT-HAUL FARES

The revenue feature is usually the one which receives the most consideration in determining whether or not a permit should be granted to operate a motor bus in direct competition with urban and inter-urban electric railroads. On account of the fact that a uniform flat rate of fare is being generally charged on street railway lines and that passengers are carried long distances as well as short distances for the same fare, the street railway finds its revenues seriously depleted if its short-haul passengers are taken away from it by a motor bus and the rate of fare for the other passengers may have to be increased, or at least it will make it impossible to reduce fares. For this reason public utility commissions in their decisions in such cases have taken the stand that street railways were entitled to protection up to a certain point. The loss in earnings of a street railway brought about through the competition of motor bus operation would result in preventing the street railway from securing the needed capital for extensions and betterments and the public would finally suffer. The check upon unwise competition in the opinion of the New York commission is abundantly justified from the point of view of public advantage. *Petition of Gray* (*N. Y. 2d Dist.*) *P. U. R. 1916A. 33.* That same commission holds that motor busses should be confined to streets and neighborhoods which are not served by street railroads except in cases where existing street railroads refuse to or can not supply the service requirements. *Petition of Gray* (*N. Y. 2d Dist.*) *P. U. R. 1916A. 33.*

In most cases it has been held that under the conditions existing in the larger cities motor-bus service can not be substituted wholly for street-car service because it would inadequately meet the public needs. At the time when the Oregon commission was considering an application for an increase in street railway fares in the city of Portland a suggestion was made that motor-bus transportation could be introduced and thus do away with street railway operations. After making a comprehensive study of the practicability and economy of motor-bus transportation the commission concluded that in these grounds it would not be justifiable to make a substitution. "Even on the more profitable lines of traffic," the commission said, "it appears that the cost would equal, if not exceed, that of the street railway, and because of the narrowness of Portland's streets the congestion of traffic which would be occasioned thereby would make it impracticable if not prohibitive." *Re Portland Railway, Light & Power Co. (Oreg.).*

According to the New York commission there is justification in the competition between a line of motor busses and a street railway line "where it appears that there is such a divergence of routes and so much greater convenience afforded by the stage route that it may fairly be said that it supplies a want of the public not already adequately met." *Re Troy Auto Car Co. (N. Y. 2d Dist. P. U. R.) 1917A 700.* Just what distance there must be between the route of the motor-bus and street-car line so that there will not be any direct competition is a matter which must be determined in each individual case. It has been held that an auto-bus line operating over a route which was 2,500 feet from the nearest street railway line did not improperly interfere with it and under those conditions a certificate was granted. *Re Woodlawn Improv. Transportation Corp.* (*N. Y. 2d Dist.*) *P. U. R. 1916D 1.*

COMPETITION OF MOTOR AND RAILROAD

When the operation of common carrier motor vehicles comes into competition with paralleling steam railroads practically the same policy is followed as in the case of competition with street railways. Where the service of the steam railroad is inadequate and unsatisfactory permits are granted. *Re King. (Calif.) P. U. R. 1919F 377.* Not many cases of this kind have arisen because the service of the motor-bus lines is in most instances purely local in character.

Commissions have uniformly held that where motor-bus lines are operating and giving satisfactory service they should be protected from competition of other operators. *Re Automobile Stage Line (Ariz.) P. U. R. 1918B 292.* If it appears, however, that there is enough business to warrant the establishment of a second line and the line already operating does not

[7] Re Troy Auto Car Co. (N. Y. 2d Dist.), P. U. R. 1917A, 700.

properly serve the public, certificates of public convenience and necessity are issued to applicants authorizing competitive service.

Where there is an unoccupied field and there are several applicants seeking permits to operate and it appears that the prospective business warrants only one line of motor busses, the commission has to take into account the relative ability of the applicants to furnish the required service. *Re King (Calif.) P. U. R. 1917F 377; Re Chicago Motor Bus Co. (Ill.) P. U. R. 1918C 320.* Such considerations as the financial responsibility of the applicants, the type of equipment, the proposed rates and schedules are all matters which are given due weight.

Through regulations and the promulgation of rules the State commissions supervise the operations of motor busses; speed limits are established, provisions are made for all kinds of safety measures, rules of conduct and the hours on duty of drivers are definitely prescribed, the capacity loads are fixed, and all manner of precautions are taken for the benefit of the traveling public.

In none of the formal proceedings have any definite principles been established as to the maximum rate of return which should be allowed to this new class of common carriers and public-utility enterprises. The business is new, and very little authentic information has been collected which would indicate what the return should be. The capital requirements are not so large as in other kinds of public utility undertakings. At the same time the business has been conducted in many instances by individuals who keep few accounts and who, at the best, have but a limited conception of the intricacies of cost accounting. Rate cases as such have not found their way to the dockets of public utility boards and commissions. Because of the lack of rate controversies the valuation of the property of these carriers has not been necessary.

The regulatory laws of the various States provide that annual reports shall be filed in accordance with the requirements prescribed by the commissions. These reports at present contain fairly accurate data as to revenue; when it comes to a classification of operating expenses there is an evident weakness. The item of depreciation usually receives very little attention; there is also noted a confusion in the minds of many of the small operators between wages and salaries on the one hand and return on the other. In those States where street railway companies have, either directly or indirectly, gone into the business of operating motor busses to supplement their rail service, reports and financial statements are filed which are properly made out by the accountants and clerks familiar with public utility cost accounting systems.

According to the information obtained from a number of State commissions which regulate the operations of all common carrier motor vehicles there is a tendency for the individual operators to be taken over by corporations organized for this purpose; consolidation of small operators is also noted. Large companies are appearing which control the routes in a given section of a State. The motor-bus business may pass through the same consolidation movement which took place in the early days of the street railway business.

TRAFFIC CONTROL AND SAFETY

(Continued from page 9)

tinuous routes east and west, and the even numbers to similar routes north and south. A traveler from the Atlantic seaboard to the Pacific would know that he was going west so long as he continued on an odd-numbered route, even though he had no map. The odd numbers would take precedence over the even numbers where a diagonal or cross route was used by both the north-and-south and east-and-west route.

Guide sign used in Pennsylvania giving direction and distance

With the aid of a simple diagrammatic map of the system, this arrangement would greatly facilitate travel in all parts of the country with assurance against loss of time or distance.

Beginning with number 100 other routes of the road system could be numbered in each State independently. A traveler on a three-digit route would know that he could not depend on an interstate connection indefinitely across the country.

Some States already have authority to give final designation to the State roads and are awaiting a decision in this matter before proceeding. This plan will be devised to make it possible to eliminate the confusion of multiple numbering and marking now so common. There is at least one road in the country, in a State that has made one of the most successful efforts at designating the State system as such that carries to-day eight different designations because it is a part of eight interstate and State routes. This confusion should and can be eliminated.

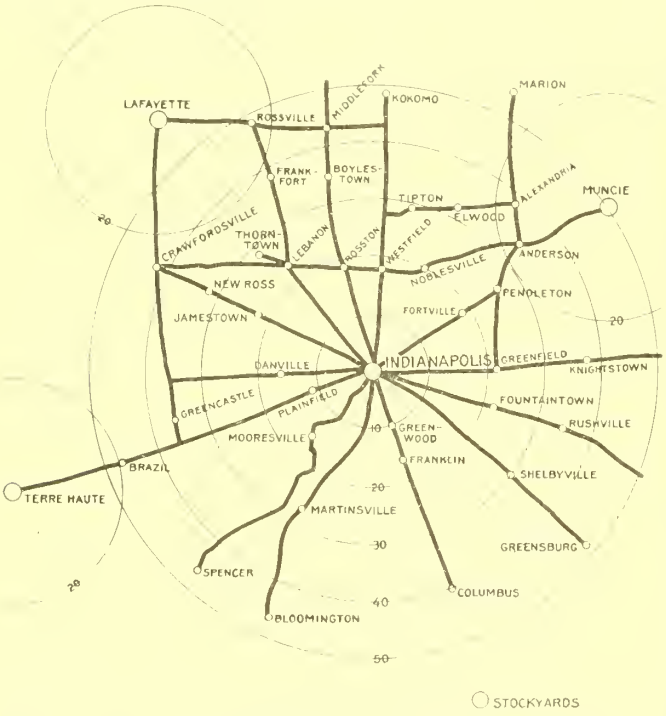
The foregoing brief and general statement of what has been done and of what is being done to promote traffic control and safety on our highways clearly discloses several facts. We have a definite and reasonably complete knowledge of principles and have a means of securing further data. We see that many excellent devices and practices are in use in many of the States, and that were it possible at once to select from each State the best details of design, the best system of marking, the most efficient policing, and to adopt the best traffic regulations already in force, we could construct a code of traffic control and of highway engineering practice that would furnish an almost complete physical and regulatory solution to the problem of securing safety on the open road. The present immediate problem is to secure uniformity and harmony in the adoption of the best practices.

TRANSPORTATION OF HOGS BY MOTOR TRUCK

BUREAU OF PUBLIC ROADS STUDY SHOWS EXTENT OF PRACTICE IN INDIANAPOLIS MARKET

By E. L. BROWNE, Agricultural Economist, U. S. Bureau of Public Roads

HIGHWAY transportation has to-day come to be one of the main pillars of our economic structure. We have seen this development taking place before our eyes during the past ten or fifteen years but we have been able only to sense its proportions as we have seen the streams of trucks passing over the highways carrying the products of farm and factory. There is a real need to know whence these trucks come, where they go, what they carry, and what it costs in order that we may form some sort of estimate of the value of the services they render.



Map showing area from which hogs are trucked into Indianapolis and location of competing markets

The transportation of livestock over the highways to markets where they are butchered has become quite general in the production areas close to such markets, and Indianapolis has been selected for study as sufficiently representative of such practice. In this area a very large proportion of the livestock produced within 50 miles of the city is trucked in and this practice has grown up where a steam and electric service equaled in few agricultural areas has existed for several years, more than twenty such lines entering the city from all points of the compass.

The use of the motor trucks for delivering livestock from the farm to the Indianapolis stockyards began in 1912 and since that time has increased rapidly. In 1913, the first year truck receipts were recorded separately; they amounted to 4.55 per cent of the total, while in 1923 the truck receipts of hogs, cattle, calves, sheep, horses, and mules amounted to 31.7 per cent of the total.

The Indianapolis stockyards, which are the largest east of Chicago, are primarily a hog market. For this reason this discussion will be confined to this class of livestock. The annual receipts of hogs transported by motor truck give an interesting picture of the rapid change which has been taking place since 1913. The volume of such receipts has become so large in recent years that the stockyards company have found it necessary to establish a separate division for trucked-in stock, known as the truck division. The following table gives the official figures for the receipts of hogs from 1913 to 1923 as reported by the auditor of the Belt Railroad & Stockyards Co.:

Receipts of hogs, Indianapolis stockyards, 1913–1923

Year	Total receipts	Truck receipts	Per cent of total
1913	1,994,624	90,821	4.55
1914	2,099,787	96,591	4.60
1915	2,435,319	136,447	5.60
1916	2,576,611	173,191	6.72
1917	2,350,730	271,994	11.57
1918	2,749,976	462,313	16.81
1919	2,936,493	711,212	24.21
1920	2,896,894	791,988	27.33
1921	2,694,705	808,595	30.00
1922	2,266,551	734,280	32.39
1923	2,875,648	934,960	32.54

TERRITORY SERVED BY TRUCK

The localities from which hogs are shipped by motor truck and distances from market vary somewhat according to road conditions and seasons. The normal trucking range is within a 50-mile radius of Indianapolis. Ninety per cent of the hogs received from this area are delivered by motor truck. There are scattering shipments from the territory 75 to 100 miles away but normally these shipments move by rail. The condition of the unimproved roads varies greatly with the season of the year. During the late winter and early spring months few haulers will accept shipments originating off the State or county roads, but during the summer and fall months livestock trucks go anywhere. With the number of miles of good roads constantly on the increase in the territory surrounding Indianapolis, the trucking radius is being constantly pushed back to the economic limit of truck haul on improved roads.

While the prevailing rates favor rail shipment for long distances there is another reason why 50 miles is about the limit of livestock hauling, that is the competition of other stockyards located as shown on the accompanying map. Stockyards of fairly large proportions are located at La Fayette, Terre Haute, and Muncie. These markets as a rule take up only local stock most of which is trucked-in.

LIVESTOCK SHIPPING RATES

A satisfactory rate must be one which is low enough to attract business in the face of competition and high enough to yield a reasonable profit. As the business is handled almost entirely by individual truck owners engaged in small scale operation without any central organization there has been no attempt to establish uniform rates. Usually the rate is agreed upon at the time shipment is made, taking into account competitive rates, number, and kind of stock, length of haul, and condition of roads. The following table shows the comparative rail and truck rates for several representative shipping points near Indianapolis.

Truck and rail rates on hogs into Indianapolis

Shipping point	Distance	Rate per hundred pounds				
		Truck	Steam railroad		Traction line	
			Single deck	Double deck	Single deck	Double deck
	Miles	*Cents*	*Cents*	*Cents*	*Cents*	*Cents*
Danville	20	20	14	12	13	11
Franklin	21	20	14½	12½	13½	
Lebanon	27	25	15	13	14½	12
Thorntown	30	30	16½	14½	14½	12
Martinsville	30	30	16½	14½	14½	12
Columbus	41	40	18	16	17	
Crawfordsville	45	45	18½	16	17	14
Greensburg	48	50	18½	16	18	
La Fayette	68	70			19	18

If every farmer had a full carload of hogs to ship at one time, railroad or traction lines offer, in practically all cases, cheaper rates than the truck haulers. However, there are relatively few farmers who have ready for market at one time a sufficient number of hogs to make up even a minimum carload. Thus the cost per head is often higher by rail, especially when one considers the barnyard to stockyard service which the truck furnishes.

For example, the minimum rate on a single-deck car over the traction line from Lebanon to Indianapolis, a distance of 27.7 miles, is $24.65 (17,000 pounds minimum weight at 14½ cents per hundred pounds). In addition it costs $5 to $8 to hire a truck to haul stock to the local shipping point. A truck which hauls 20 head of hogs weighing 300 pounds a head, charges 25 cents per hundred pounds or $15 for the load.

In cases of this kind the motor truck offers the cheaper transportation and would be chosen by the farmer even were no other advantages offered as is often the case. In the above sample rates it is seen that the truck rates are approximately proportional to distance while the steam railroad and traction line rates increase at a less rapid rate with added distance. When the cost of hauling a truck load of hogs approaches the rate for a minimum carload there is an incentive to pool shipments and accept the trouble of securing cars, loading, marking hogs, or erecting partitions to distinguish ownership in order to secure a cheaper transportation charge.

Charges for truck hauling are paid directly to the truck operator by the commission firm handling the consignment, a corresponding deduction being made from the the shipper's check. In this way the hauler is always sure of his money and is not bothered by having to make the collection, while the shipper is saved the trouble of making payment.

OWNERSHIP AND OPERATION OF LIVESTOCK TRUCKS

The trucks engaged in livestock hauling are owned and operated by men living in the country or small villages in the rural sections. The owner usually makes trucking his sole business but in some cases he may farm a few acres during spare time. There are no fleet organizations operating into the Indianapolis stockyards. Most haulers operate but one truck; now and then one runs two or three trucks. There is an increasing number of farmers who own and operate trucks of their own. At the present time there are no cooperative farmers' organizations operating trucks.

To operate successfully a livestock hauler must keep his truck in use the greatest possible number of days during the year. When there were comparatively few trucks operating it was not uncommon for trucks to average 300 to 325 trips a year; of late so many have entered the business that the average number of trips is being rapidly cut down and the profits of operators reduced. Few operators realize that their fixed operating costs go on whether their trucks are used or not. With competition among haulers increasing yearly many haulers have had to look for other sources of income.

THE RETURN LOAD

The return load offers an extra income which in many cases may be the margin of profit for the year. There are three types of return load carried. First, there are the loads of feeder stock which are picked up at the yards. This class of return load is very profitable as no time is wasted in making up a load and it is unnecessary to clean out the body before reloading. Second in importance is the hauling of feed, which has increased considerably since the Farm Bureau has located its warehouse near the stockyards entrance. The body of the truck is thoroughly cleaned and washed before the feed is loaded. General merchandise is least important and least desirable due to the time lost in collecting a load, as shipments are usually small and scattered over the city. In order to stimulate the return-load movement the rates charged approximate one-half of those charged for the trip to the city.

A central depot where freight could be collected and consigned has never been worked out as there is very little cooperation between various haulers. Such a depot could easily be operated by one man whose duty it would be to receive and route the freight consigned to the depot and it would be of benefit to all parties concerned.

TYPES OF TRUCKS AND BODIES

The popular capacities are the 1, 1½, and 2-ton trucks with very few heavy-duty types in operation. These smaller types are better suited to the average loads of hogs which are shipped than are the larger capacities. The use of trailers rather than large trucks seems to be popular with many haulers.

Solid tires are most generally used. There are, however, many haulers operating in the part of the territory in which the roads are unimproved who prefer pneumatic tires on account of the advantage during the winter and spring months.

The bodies are the regular platform rack, of which the larger types are easily converted into a double-deck arrangement. A few of the large double-decked, 3-ton trucks will haul 40 to 50 head on one load. The table following gives the carrying capacities of various sizes of trucks.

Number of hogs loaded on trucks of various capacities

Capacity in tons	Number of head of 200-pound hogs
1	12
1½	18
2	25
2½	30
3	35
3 (double-decked)	40-50

NOTE.—A single-decked railway stock car will load 70 to 80 head while a double-decked car loads 140 head (200-pound hogs).

During the year 1923, 71,000 truck deliveries, totaling 934,960 hogs, were made, or an average of 13 hogs per truck load.

TRUCK TRANSPORTATION OFFERS ADVANTAGES TO FARMERS

Shrinkage in truck shipment does not vary greatly from rail shipment when the time element in each is the same. Hogs which are trucked in usually show less loss in weight because they have been in transit a shorter time. This does not necessarily mean that the farmer will get more for his hogs than those that have have been delivered by rail. The difference in weight in the two shipments is the difference in "fill" retained by the hogs after shipment. There is no actual difference in the dressed weight of the hogs. Commission men usually know how long shipments have been in transit and are in a position to judge how much a carcass will dress and what they should pay for it.

While no definite figures are available as to the number of "deads" received by the various means of transport, yet it is well known among producers and commission men that there are fewer losses by truck. Trucked-in hogs move by night during warm weather, whereas rail shipments are often loaded and shipped during the heat of the day. According to figures compiled by the United States Department of Agriculture, 1 out of every 319 hogs shipped by rail dies in transit. It is seldom that one sees a dead hog in front of the unloading chutes of the truck division.

Another advantage is that hogs of one shipper are seldom mixed with those of another shipper, which eliminates fighting among strange hogs. While in transit the driver is always on hand to prevent smothering. He also acts as the representative of the shipper at the yards upon arrival of the truck.

Typical trucks used in hauling hogs to the Indianapolis market

The average hog raiser who is fattening 50 to 60 head always finds that some animals fatten more readily than others. The truck can be used to ship those hogs which are ready for slaughter while the others can be held back. To hold the other hogs over this period would be economic loss; it is also the extremely fat hog that dies in transit.

Before the advent of good roads and the motor truck many raisers who marketed but 20 to 30 head sold to a local buyer who assembled car lots and who traded on a wide margin. The local buyer not knowing exactly when he would move his newly acquired stock generally paid 75 cents to $1.25 below the market price. The raiser had also to deliver the stock to the point at which the local buyer planned to assemble the car. This system has entirely changed. The local buyer is at present eliminated within the trucking radius and the producer ships directly to the yards where his stock is sold at the market price.

A glance at the map will show that many farmers live within trucking distance of two or more stockyards. The motor truck allows a farmer to take advantage of whatever yard offers the best price. It is not uncommon for the small stockyards of Lafayette, Muncie, or Terre Haute to become oversupplied, while at a more distant point, such as Indianapolis, the market might be brisk.

A trailer with a cattle rack body used for hauling a few head of livestock

A close study of market prices and tendencies enables many farmers within trucking radius to take advantage of rises in market prices. The farmer by means of a small radio receiving set can catch the market openings and still have plenty of time to truck his stock to the yards before closing time. Many shippers have found it extremely difficult to get cars when the market offers the best opportunities. However, if the farmer owns a truck or can hire one on short notice, as is generally the case within the Indianapolis trucking area, he is in position to take advantage of every opportunity.

That trucked-in lots are often more uniform and sent forward at the most opportune time so that they average a somewhat higher price is shown by the following table which gives the highest sales price for hogs at the Indianapolis stockyards for the month of March, 1924:

Highest daily sale price for hogs at the Indianapolis market during March, 1924

Date	Delivered by rail	Delivered by truck	Date	Delivered by rail	Delivered by truck
1	$7.60	$7.65	18	$7.70	$7.70
3	7.65	7.65	19	7.70	7.75
4	7.70	7.75	20	7.70	7.75
5	7.85	7.85	21	7.85	7.90
6	7.75	8.00	22	7.80	7.90
7	7.75	7.85	24	7.90	7.85
8	7.75	7.85	25	7.65	7.75
10	7.75	7.75	26	7.70	7.70
11	7.90	7.85	27	7.85	7.75
12	7.85	7.90	28	7.75	7.75
13	7.70	7.75	29	7.70	7.75
14	7.65	7.65	31	7.85	7.90
15	7.75	7.70			
17	7.85	7.75	Average	7.756	7.784

HIGHWAY TRANSPORTATION SURVEYS UNDERWAY IN COOK COUNTY, ILLINOIS, AND MAINE

TWO highway transportation surveys similar in nature and scope to the survey now in progress in Pennsylvania (PUBLIC ROADS, June, 1924) have recently been started. A State-wide survey in Maine conducted by the Bureau of Public Roads in cooperation with the State Highway Commission was begun on June 30 and will continue for a period of four months. Data similar in nature to the information being obtained in Pennsylvania and described in the June issue of PUBLIC ROADS are being recorded at 95 observation stations so located as to secure traffic data on all important highways in the State. At 20 stations complete weight information on all motor trucks, as well as complete information on all other types of traffic, is being recorded, the weight information being obtained by the use of portable scales. At the remaining 75 stations complete data, except weights, are being recorded on all types of traffic. Data at all stations are being recorded in such a manner that weights secured at the weighing stations can be imputed to traffic at stations which have no weighing facilities.

Observations are recorded by personnel specially trained for the purpose and organized into parties, each having a certain number of stations to operate. The schedules of these parties are so arranged as to obtain complete information on each station at all hours of the day and night and on each day of the week. Two parties are organized to operate the weight stations, five parties to operate recording stations (stations that do not have weighing facilities) and two parties to operate selected stations during the night hours.

The survey in Cook County, Ill., which is being conducted in cooperation with the Cook County department of highways began on July 7, and will also operate for a period of four months. This survey is similar in purpose and scope to the Pennsylvania and Maine surveys with such modifications as are necessary to provide the data required to solve the special problems of highway development in an area contiguous to a large city. Data will be recorded at 73 stations in Cook County supplemented by 14 stations located within the city of Chicago.

At 17 of the stations in Cook County complete weight information is being recorded on all motor trucks, weights being obtained by the use of portable scales and complete data on all other types of traffic. At the remaining 56 stations complete data with the exception of weight information are being recorded on all types of vehicles. Information is recorded in such a manner that weights obtained at weighing stations can be imputed to the traffic observed at recording stations. At the stations located within the city of Chicago the density of various types of traffic is being recorded.

Approximately 30 men divided into nine parties are employed in recording data. Two parties have been organized to operate the 17 weight stations, four parties to operate 56 recording stations, one party to operate the stations located within the city of Chicago, and two parties to operate selected stations during the night hours. The schedule of these parties is so arranged as to obtain complete records of traffic at each station at all hours of the day and night and on each day of the week.

Investigations are also being made of the effect of traffic density upon the speed of traffic; of the effect of curves, railway crossings, and highway intersections upon traffic congestion; and of the effect of traffic density upon the point of application of wheel loads with reference to the edge of the pavement and the shoulders of the roadway.

A special investigation of motor trucking in the Chicago area is also being conducted. This investigation will include a study of the extent and nature of motor trucking in this area, the organization and operating methods of trucking companies, trucking rates, operating costs, the use of motor trucks in terminal areas, suburban and intercity transportation of freight by motor truck, the nature and extent of competition between highway transportation and rail and water transportation of freight and passengers, and of marketing by motor truck. This study is being made by special investigators who will interview truckers and shippers throughout the area and gather data on this problem from all available sources.

C. A. HOGENTOGLER TO SERVE WITH ADVISORY BOARD ON HIGHWAY RESEARCH

C. A. Hogentogler, of the United States Bureau of Public Roads, has been granted leave of absence in order to conduct for the Advisory Board on Highway Research of the National Research Council a fact-finding survey of the economic value of reinforcement in concrete pavements. This survey is to be national in scope, and will be conducted in cooperation with interested agencies. It is proposed to cover the various soils, traffic, and climatic conditions throughout the United States.

Mr. Hogentogler has had 15 years of experience in highway construction and highway research which well qualifies him to take charge of the present investigation. After graduation from the Pennsylvania State College, and a short period with the Pennsylvania Steel Co., he was with the Pennsylvania State Highway Department, followed by several years on street and road construction with the borough of Columbia, Pa. For two years he was assistant professor of civil engineering at the University of Idaho. He was then engaged in research with the United States Bureau of Standards, and finally with the United States Bureau of Public Roads, in which organization he has been for the past six years.

During this period Mr. Hogentogler has been actively engaged in a number of important highway researches. These include the first impact and wear tests at the Arlington experiment station and the tests to determine the cushioning properties of tires now being conducted by the United States Bureau of Public Roads in cooperation with the Rubber Association of America and the Society of Automotive Engineers. In 1923 Mr. Hogentogler, as representative of the Bureau of Public Roads, conducted the study made in cooperation with the Advisory Board on Highway Research which resulted in the publication by the National Research Council of its Bulletin No. 35, entitled "Apparatus Used in Highway Research Projects in the United States."

Mr. Hogentogler is the author of a number of important research papers which have appeared in PUBLIC ROADS and which have been reprinted in various technical periodicals. He is a member of the American Concrete Institute and an associate member of the American Society of Civil Engineers.

CONCRETE TEST ROAD IN PENNSYLVANIA

The Pennsylvania State highway department and the United States Bureau of Public Roads are cooperating in the construction of a concrete test road located on State Route No. 50 in the vicinity of Harrisburg, Pa. The project selected is approximately 1,700 feet in length and its site is on a silty loam soil through swampy land.

The purposes of the test road are as follows:

1. To determine the action of a concrete pavement with a 5-inch center thickness and 8-inch edge thickness (8–5–8 section) under a 26,000-pound gross load.
2. To determine the influence of subgrade material on the strength of the pavement.
3. To obtain further data on balancing the cross-section design.
4. To determine the effectiveness of the tongue-and-groove type of longitudinal joint.

Six different types of subgrades have been prepared. They are as follows: Old macadam road scarified and rerolled, sandy loam, shale, clay, clay overlaid with 4 inches of sand-clay mixture and clay overlaid with 4 inches of cinder-clay mixture.

Concrete construction is being carried out in the usual manner, and at approximately the center of each subgrade section a transverse expansion joint is being provided. Adjacent to this expansion joint and on one side of the longitudinal joint over a length of 20 feet, inserts are being placed in the slab in such a manner that strain gauge readings may subsequently be obtained both for the top and bottom of the slab.

Control specimens, consisting of beams and cylinders, representative of the particular portion of the pavement in which the strain gages are to be installed are being made. These specimens are to be tested and broken at the ages of 15 days, 28 days, 6 months, and one year, for the determination of the modulus of elasticity, the modulus of rupture and the compressive strength of the concrete.

The pavement when completed will be subjected to heavy truck loads up to 26,000 pounds gross. Stress readings will be obtained with the truck run near the edge and at the center of the road. The truck will be run during the summer and during the time of the spring thaw. Field determinations of subgrade moisture and bearing value of the soil will be made on the days stress readings are obtained.

From the results of the above tests it is hoped that answers will be supplied to the following questions:

1. Is an 8–5–8 section adequate for occasional 26,000-pound traffic?
2. What influence has the subgrade on the strength of the slab?
3. Will a 4-inch layer of good material improve the strength of a concrete road when laid on a poor subgrade material?
4. Is an 8–5–8 section a balanced design or should the center and edge thicknesses be adjusted?
5. How much of the load does the tongue-and-groove type of longitudinal joint transmit from one side of the slab to the other?
6. If an 8–5–8 section is not safe under a 26,000-pound load, what is the safe load under spring conditions and under ordinary conditions?

COMMITTEE ON TESTS AND INVESTIGATIONS, A. A. S. H. O., MEETS AT BUREAU OF PUBLIC ROADS

A meeting of the Committee on Tests and Investigations of the American Association of State Highway Officials was held at the Bureau of Public Roads on June 23 and 24, 1924. The following States were represented at the meeting:

Delaware, Robert Schmidt, testing engineer.
Florida, Harvey A. Hall (represented by R. L. Jenkins).
Illinois, H. F. Clemmer, engineer of materials.
Iowa, R. W. Crum, engineer of materials and tests.
Kansas, C. H. Scholer, engineer of tests, Mr. Dolly.
Maryland, Dean A. N. Johnson.
New Hampshire, W. F. Purrington, chemist and testing engineer.
New Jersey, R. B. Gage, chemical engineer; J. G. Bragg, testing engineer; Mr. Baughmann.
North Carolina, R. L. Oberholzer.
New York, J. E. Myers, chemist; W. L. Blaum, representing Mr. Treadwell.
Ohio, A. S. Rea, engineer of tests.
Pennsylvania, H. S. Mattimore, engineer of materials; M. H. Ulman, assistant engineer of materials.
Tennessee, O. H. Hansard, chemical engineer.
Virginia, Shreve Clark, assistant engineer.
West Virginia, R. B. Dayton, materials engineer.

The Bureau of Public Roads was represented on the committee by Messrs. Smith, Milburn, Anderton, and Jackson.

The committee discussed and prepared tentative specifications for various types of bituminous and nonbituminous road materials. Among the materials considered were cement, stone, gravel and slag, sand as fine aggregate, reinforcing steel, culvert pipe, wire rope, asphalt, tars, bituminous premolded joints, and paint. After specifications for these materials have been finally adopted by the committee it is planned to present them to the American Association of State Highway Officials through its committee on standards for final action.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JULY 31, 1924

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 189,707.91	$ 94,229.55	36.9	$ 15,163,939.72	$ 7,415,796.02	843.8	$ 287,894.71	$ 114,403.64	7.0	$ 1,442,266.25	Alabama
Arizona	8,336,365.41	4,267,683.88	527.8	249,398.74	124,699.37	16.3	2,126,052.16	1,269,917.23	148.5	266,850.29	163,072.20	32.7	1,630,328.32	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	122,739.41	60,850.67	4.9	6,096,476.87	1,965,651.70	336.0	1,997,568.97	945,362.19	81.2	1,676,169.91	Arkansas
California	12,999,075.03	5,647,148.17	633.7	750,456.11	396,623.78	25.1	12,754,895.61	6,803,636.77	439.8	$1,743,516.22	869,300.62	38.3	3,377,594.66	California
Colorado	8,108,070.31	4,029,698.97	502.6				5,387,187.51	2,905,223.07	191.6	444,999.28	249,731.34	23.4	2,375,027.62	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	199,024.56	98,423.00	4.9	2,839,677.86	930,529.90	47.3	95,905.51	25,301.25	1.6	1,057,382.25	Connecticut
Delaware	3,066,832.22	1,007,714.83	72.6				1,096,672.50	420,933.60	27.6	456,964.00	164,700.00	11.0	146,161.57	Delaware
Florida	961,134.07	461,470.92	48.8				9,190,174.82	4,506,551.34	259.3	398,642.71	171,123.94	10.2	1,147,740.60	Florida
Georgia	17,167,373.32	7,955,805.20	1214.2	245,795.65	122,013.76	18.6	8,894,094.72	4,303,227.15	720.1	1,496,002.68	688,913.25	115.5	1,379,937.64	Georgia
Idaho	8,161,697.92	4,092,395.52	506.8	82,326.24	46,617.62	0.1	1,452,536.71	641,613.01	101.2	1,006,048.61	548,146.93	59.7	1,048,536.92	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	493,620.15	244,171.61	14.6	16,300,530.37	8,099,672.51	553.0	303,482.87	151,741.43	11.5	2,661,359.92	Illinois
Indiana	7,577,444.16	3,655,640.97	225.7	162,566.05	89,656.72	6.4	15,982,526.45	7,801,426.22	522.8	634,177.67	307,774.89	19.8	2,457,991.20	Indiana
Iowa	23,195,778.19	9,237,031.86	1682.9	98,400.80	43,700.00	2.6	8,576,488.76	3,982,957.08	620.3	384,200.75	187,242.39	43.6	1,885,205.67	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	2,781,749.75	932,516.99	73.9	14,802,618.63	5,843,065.89	575.6	2,132,084.04	913,632.51	105.9	1,506,593.61	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4				7,594,366.28	3,663,928.55	329.2	101,082.14	50,541.07	4.0	2,043,322.10	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	15,207.25	7,603.62		5,712,124.28	2,829,358.51	372.3				792,336.51	Louisiana
Maine	6,911,068.78	3,299,935.38	230.7	198,869.11	98,755.62	9.4	1,495,468.85	743,302.17	53.9	479,738.34	231,798.78	21.6	716,160.05	Maine
Maryland	6,760,044.42	3,213,321.78	243.2				2,260,474.84	928,085.95	72.2	648,061.15	250,366.54	18.4	267,175.73	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8				5,264,835.09	1,832,245.47	92.5	813,848.46	213,917.12	11.8	1,767,890.19	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5				13,928,561.23	6,708,729.87	494.7	45,171.00	22,585.50	1.4	3,087,844.40	Michigan
Minnesota	24,037,561.24	9,865,843.07	2292.0	108,570.20	39,600.00	12.2	9,227,309.72	4,265,429.97	746.6	1,140,925.17	565,100.00	185.0	562,545.96	Minnesota
Mississippi	7,888,193.69	3,828,941.39	655.0				7,706,383.19	3,845,261.10	489.8	485,474.59	242,537.28	10.3	1,614,533.23	Mississippi
Missouri	11,352,027.70	5,245,899.15	803.5				15,480,902.51	7,386,933.92	647.5	3,875,845.78	1,875,736.82	218.3	3,431,618.08	Missouri
Montana	8,867,279.16	4,364,335.12	791.4	8,089.35	8,089.35		2,964,964.63	2,169,910.66	281.8	1,079,275.33	612,493.90	119.2	3,771,587.97	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	68,571.72	34,285.86	5.9	6,909,749.52	3,361,602.38	720.2	1,032,793.23	511,216.75	84.6	3,829,149.42	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	365,472.06	318,584.01	15.6	4,464,114.49	3,733,529.62	378.5	67,255.43	56,088.90	3.5	928,493.49	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3				981,701.45	461,970.62	33.2	375,706.04	175,700.27	10.8	309,425.53	New Hampshire
New Jersey	7,623,796.12	2,661,531.40	148.7	217,382.46	76,260.00	3.8	10,705,257.33	2,805,782.10	75.2	417,573.68	107,667.49	4.1	938,005.92	New Jersey
New Mexico	6,306,286.45	2,758,849.68	714.3				6,568,161.27	4,314,425.58	684.2	812,968.30	499,975.47	51.6	1,016,081.27	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	254,056.13	99,430.84	9.4	24,941,766.96	9,693,252.97	569.1	7,684,000.00	2,455,653.75	152.3	6,201,966.00	New York
North Carolina	12,567,732.97	5,676,757.66	884.7				9,623,594.36	3,484,417.74	266.6	2,354,959.10	1,060,134.58	55.2	2,072,941.02	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1587.9	86,740.75	43,370.37	16.0	3,370,975.83	1,660,123.46	539.8	381,776.20	190,888.07	60.2	2,060,768.68	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	459,731.22	163,826.34	13.1	12,385,183.65	4,717,444.78	340.2	2,750,393.94	935,045.01	76.0	2,443,927.88	Ohio
Oklahoma	12,986,865.26	5,888,862.03	497.3	411,850.09	187,247.45	18.2	8,493,819.45	4,119,841.77	368.3	2,028,739.97	865,631.10	95.0	1,475,070.65	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	79,706.63	48,435.61	7.9	2,642,204.41	1,656,086.90	167.7	872,730.10	533,558.61	45.4	448,984.09	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	197,131.55	73,986.25	4.6	19,620,514.18	5,637,487.50	335.0	4,813,903.07	1,298,345.00	85.3	3,477,102.46	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	157,932.08	62,600.00	3.1	1,243,889.16	436,306.80	25.2				652,906.24	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	240,520.39	117,569.88	50.6	5,955,771.01	2,344,763.35	464.3	519,748.19	233,683.58	61.9	867,463.97	South Carolina
South Dakota	8,674,597.56	4,244,636.27	989.8	103,810.58	51,905.28	12.4	7,237,602.57	3,670,982.83	1002.2	513,798.37	376,425.51	72.3	374,730.11	South Dakota
Tennessee	6,805,683.36	3,313,936.07	259.6	554,569.74	427,434.67	24.0	12,824,954.12	6,028,526.47	403.3	1,642,967.93	710,195.61	62.6	1,544,541.98	Tennessee
Texas	42,341,998.56	16,190,624.91	3122.8	726,947.57	346,567.70	66.3	25,249,202.33	9,364,108.01	1564.7	3,471,735.76	1,249,038.73	236.6	3,953,873.65	Texas
Utah	3,304,423.76	1,835,605.92	219.0	138,485.53	101,152.07	23.7	4,245,735.17	2,705,463.91	280.5	1,036,761.36	701,792.94	69.6	712,258.16	Utah
Vermont	1,922,114.16	942,769.12	74.4				1,527,160.47	731,432.50	42.6	392,931.62	194,701.40	7.9	665,075.98	Vermont
Virginia	10,035,301.48	4,801,792.43	562.5				9,133,669.60	4,206,638.29	310.6	1,688,147.75	562,716.86	67.7	1,021,815.42	Virginia
Washington	11,384,615.67	5,290,895.45	457.0				2,851,003.81	1,306,000.00	121.6	903,139.27	415,300.00	33.8	874,482.55	Washington
West Virginia	5,489,747.95	2,365,041.63	255.6	466,127.14	211,158.74	11.7	4,315,997.16	1,947,380.61	152.1	1,079,140.03	474,528.01	35.0	756,022.91	West Virginia
Wisconsin	15,753,903.15	7,441,033.57	1325.3	266,053.95	134,026.97	9.2	4,099,336.27	2,040,340.42	179.3	586,053.18	282,155.39	15.4	3,780,894.65	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	667.6	24,652.06	16,078.54	0.9	4,464,441.26	2,718,270.41	341.8	131,821.27	69,520.00	1.4	805,053.35	Wyoming
HAWAII														HAWAII
TOTALS	$ 549,655,391.27	$ 237,852,399.82	32452.9	$ 10,860,886.93	$ 4,970,594.54	522.5	$ 386,357,594.14	$ 175,271,775.88	18341.6	$ 56,282,794.56	$ 23,625,846.62	2541.6	$ 365,625.00 83,404,383.14	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost 52,378,332.83 Federal aid 24,358,593.57 Miles 2,637.2

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 7 ▽ SEPTEMBER, 1924

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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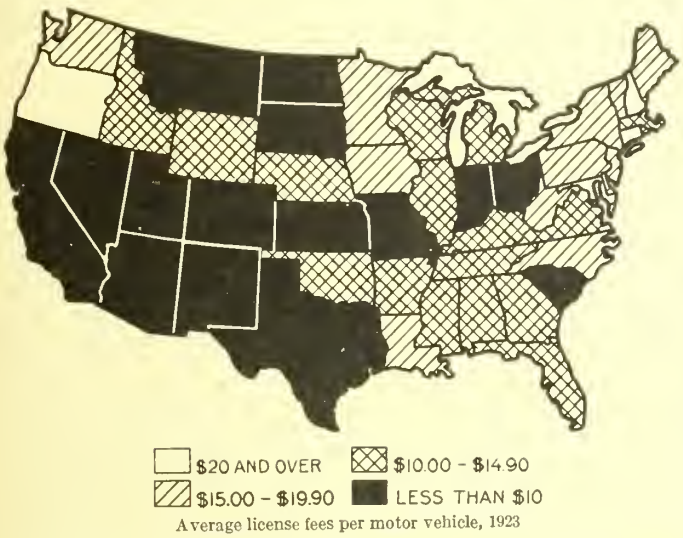
MOTOR VEHICLE FEES AND GASOLINE TAXES

A Review of the Development of Present-Day Policies in Motor Vehicle Taxation

By Henry R. Trumbower, Economist, U. S. Bureau of Public Roads

FROM 1901, the first year in which any State imposed a license tax on automobiles, to the end of 1923 owners of motor vehicles, both trucks and passenger cars, paid into the treasuries of the several States, as nearly as can be determined, $798,314,319. New York, the first State to levy the tax, collected during the first year just $954; in 1923, a little over 20 years later, the motor vehicle license fees of the country amounted to $188,970,992. The amounts collected in each year are set forth in the following tabulation, which also shows the number of vehicles registered and the average revenue per vehicle since 1909. Prior to that year it was impossible to obtain the information in regard to the number of vehicles, and not until 1912 were the registration methods of the various States sufficiently systematized to insure accuracy in the statistics.



Average license fees per motor vehicle, 1923

Total motor vehicle license fees, yearly registrations, and average receipts per vehicle, 1901–1923

Year	Number of vehicles registered	Total gross receipts [1]	
		Amount	Average per vehicle
1901		$954	
1902		1,082	
1903		26,865	
1904		33,411	
1905		62,500	
1906		192,706	
1907		334,916	
1908		484,277	
1909	294,000	938,860	$3.20
1910	472,700	2,227,434	4.70
1911	677,000	3,967,475	5.85
1912	1,010,399	5,638,878	5.60
1913	1,258,062	8,192,253	6.50
1914	1,711,339	12,382,031	7.20
1915	2,445,666	18,245,711	7.45
1916	3,512,996	25,865,369	7.35
1917	4,983,340	37,501,233	7.55
1918	6,146,617	51,477,419	8.35
1919	7,566,446	64,697,255	8.55
1920	9,231,941	102,546,212	11.10
1921	10,463,295	122,478,654	11.70
1922	12,238,375	152,047,823	12.50
1923	15,092,177	188,970,992	12.50
Total		798,314,310	

[1] These receipts do not include any gasoline taxes.

In 1909, according to the best information obtainable, about 294,000 automobiles were registered; 14 years later the registration reached 15,092,177. According to these figures the average license tax collected from each motor vehicle in 1909 was $3.20 and this average had risen by 1923 to $12.50. The yearly increase in the average per vehicle is evidence of the steady advance in the charges and fees for motor vehicle licenses which characterized the period. How this advance occurred is shown by the following table, which presents a classification of the States for each year beginning with 1913 according to the average revenues received per car:

Classification of States according to average motor vehicle revenues per vehicle

Average revenues per vehicle	Number of States in each class [1]										
	1913	1914 [2]	1915	1916	1917	1918	1919	1920	1921	1922	1923
$4.90 and less	21	17	13	14	14	6	7	2	1		1
$5–$9.90	15	17	20	21	22	27	24	17	15	15	15
$10–$14.90	9	11	12	11	10	9	10	18	21	17	16
$15–$19.90	4	3	3	3	3	6	7	10	11	14	13
$20–$24.90						1	1	2	1	1	3
$25–$29.90										2	1
$30–$34.90			1								
Total	49	48	49	49	49	49	49	49	49	49	49

[1] Includes District of Columbia.
[2] On account of the Supreme Court declaring the licensing law unconstitutional, no revenues were reported for Michigan.

As the table shows, there were 21 of the States, or 42.8 per cent, which in 1913 received average revenues per vehicle less than $5; and 36, or 73.5 per cent, had average annual receipts less than $10. From this situation in 1913 there was a steady progression by amendment and revision of the State laws involving increases in the rates of fees and classifications and reclassifications of motor vehicles, each measure tending to raise the average yield per vehicle until by 1923 the only jurisdiction in which the average fee remained less than $5 was the District of Columbia. By 1920 it is observed that over 60 per cent of the States charged fees and license taxes which yielded an average of $10 and more per vehicle, and by 1923, 33 or 67 per cent, were in this class. In the same year over a third of the States had in effect scales of license fees which resulted in average revenues per vehicle of $15 and over and four of these States charged license fees which brought the average annual revenues up to over $20 per vehicle.

Another reason for the increase in the average revenues per vehicle was the special classification of the motor truck by which many of the States established still higher rates for this type of motor vehicle. Special classifications and charges covering automobiles for hire tended to bring about advances in the average revenues.

WIDE VARIATION IN LICENSE FEES OF VARIOUS STATES

The wide variation in the amount of the fees collected by the States is illustrated by the following tabulation, from which it appears that the average fees

per motor vehicle, based on gross registration receipts, range all the way from $26.36, the amount collected by New Hampshire, to $5.73, the average in Arizona. The average fee of $4.78 received by the District of Columbia is the only one below $5. Of the 15 States whose average receipts per motor vehicle range between $5 and $10, only 3, Ohio, South Carolina, and Indiana, are east of the Mississippi River. There appears to be a tendency for the Eastern States to charge a higher scale of license fees, though there are specific exceptions to this statement. Oregon, for example, an extreme Western State, collected $24.52 as the average license tax in 1923. As pointed out in the article entitled "The Incidence of the Highway Tax Burden" (PUBLIC ROADS, vol 5, No. 4, June, 1924), the scale of motor vehicle fees is to a certain extent dependent upon the highway expenditures of the particular State and the policy which the State is following in financing its program of highway expenditure.

Average fees per motor vehicle based on gross registration receipts, 1923

New Hampshire	$26.36	Alabama	$12.17
Oregon	24.52	Tennessee	11.82
Connecticut	23.82	Nebraska	11.72
Maryland	20.90	Wisconsin	10.84
Vermont	17.79	Oklahoma	10.48
New Jersey	17.76	Wyoming	10.40
Delaware	17.22	Mississippi	10.33
Rhode Island	16.86	Illinois	10.00
West Virginia	16.52	Montana	9.88
New York	16.49	Nevada	9.80
Minnesota	16.33	California	9.64
Louisiana	16.04	New Mexico	9.21
Iowa	15.46	Kansas	9.15
Maine	15.29	Ohio	9.04
Pennsylvania	15.18	South Dakota	8.59
North Carolina	15.10	Missouri	8.43
Washington	15.10	Texas	7.91
Idaho	14.65	Utah	7.23
Virginia	14.62	South Carolina	7.08
Massachusetts	14.53	North Dakota	6.96
Michigan	14.37	Indiana	6.33
Kentucky	13.50	Colorado	5.96
Florida	12.92	Arizona	5.73
Arkansas	12.67	District of Columbia	4.78
Georgia	12.40		

An investigation of the specific license fees charged by the various States shows that, for the purpose of levying such a tax, motor vehicles are divided into several classes. The primary classification is fourfold, namely, passenger cars, commercial cars or motor trucks, trailers, and motor cycles. This classification is based upon the type of vehicle. Many States make a further differentiation and make special provision in the scale of fees for passenger cars and motor trucks used for hire or classed as common carriers. Vehicles belonging to and used by automobile dealers are also in most instances put in a special class and taxed at a rate different from, and usually higher than, the rates applied to similar cars used by others than dealers. A number of States also derive motor vehicle revenues from licenses issued to drivers of cars, and the drivers are again divided into "operators not for hire" and "operators for hire."

On January 1, 1924, 11 different methods of levying fees upon passenger cars were in effect in the various States. These ranged from the simple flat rate with no attempt at classification to complicated formulas for determining what should be paid for the privilege of operating motor vehicles of the various types. To show how this development of motor vehicle license tax systems proceeded from simple systems to the various systems now in effect there is presented below in tabular form a statement setting forth the different licensing systems in effect in the years 1908, 1912, 1914, 1921, and 1924:

Methods of licensing passenger automobiles by the various States by periods

Basis of license fee	Number of States				
	1908[1]	1912[2]	1914[3]	1921[4]	1924[5]
Flat rate	27	22	19	3	1
Horsepower	3	14	27	27	19
Weight of vehicle				5	7
Gross weight (vehicle and load)				3	6
Cost or value of vehicle				2	2
Piston displacement				1	1
Horsepower plus gross weight				3	4
Horsepower plus weight, unloaded				1	4
Horsepower plus cost				1	
Weight plus cost				1	2
Weight plus horsepower plus cost				1	1
Flat rate plus weight					1
No license law	18	12	2		
Total	48	48	48	48	48

[1] Automobilist's Legal Adviser: Nichols.
[2] Motor Car Laws of all States in the Union: Dame-Handy.
[3] Good Roads Year Book, 1914.
[4] Engineering News-Record; Sept. 1, 1921.
[5] Report, Motor Vehicle Conference Committee, 1924.

In 1908 there were still 18 States which had no registration and licensing laws, and of the 30 States which then licensed automobiles 27 had in effect flat-rate schedules and 3 charged fees which were determined by the horsepower of the car. By 1912 there were 36 States which made a charge for automobile licenses, and 14 of these based the amount of the fee upon a horsepower classification; 22 still charged flat rates. Two years later, in 1914, all but 2 of the States had adopted motor vehicle license laws; 19 of them charging flat rates and the remainder a variable fee based on horsepower. In the next seven years great changes were made, mainly by the substitution of another method for the flat-rate fee. In 1921 only 3 States continued charging flat license fees; 27 had in effect horsepower schedules and the remainder had introduced other bases to determine what was to be paid for a license. In the next three years the same trend continued and at the beginning of 1924 only 1 State was charging a flat rate and the remainder were following other methods, the horsepower type of schedule still remaining the most popular.

FROM POLICE POWER TO HIGHWAY REVENUE

As long as the emphasis in licensing automobiles was placed upon the State's police power a flat charge of a certain amount upon the registration of a car was deemed sufficient. The fee in most cases was nominal and was not regarded as a revenue-producing measure. When, however, the licensing of motor vehicles and the collection of registration fees began to be related to the problem of highway construction and maintenance the revenue aspect was brought into prominence, and as the rate was advanced classification systems were introduced which would take into account two factors. One of these factors was the extent of the road damage

assumed to be caused by different types of cars. It was felt generally that heavy cars were more destructive in their use of the road than lighter cars, and a classification according to horsepower appeared to be a simple way of obtaining greater revenues from the larger and heavier machines. The other theory which to a certain extent was evident in the introduction of classification systems was the "ability-to-pay" principle which crops out in all taxation discussions. Aside from the question of highway use and road damage, the opinion was frequently expressed in State legislatures that the owner of the larger car was in all probability better able to pay the higher license fees than the owner of the light and small car. This was an additional argument for changing from the flat-rate system to the horsepower basis for determining license fees.

The introduction of a tax on gasoline by many of the States as a means of producing revenues for highway purposes has also had some effect upon the extent to which these States have gone in working out classification schemes in connection with the levying of license fees. The economic aspects of the gasoline tax will be discussed in another place, but it should be brought out at this time that the only State which now has a flat license fee, California, changed from the horsepower basis of classification to the flat rate when the gasoline tax went into effect on the theory that the greater portion of the revenues which the State was entitled to collect to compensate for the use of the highways would be produced by the gasoline tax. At the present time California charges $3 as the annual license fee for a gasoline or steam propelled passenger vehicle, irrespective of its size, horsepower, weight, or capacity.

THE BASES OF LICENSE FEES

The horsepower basis of license fees is a classification which can be easily applied. The schedules are usually simple and leave no doubt in the mind of the taxpayer as to the correctness of the fee. The schedule is applied in several ways. Some States classify automobiles according to horsepower and charge a specific fee for vehicles coming into the different groups. North Carolina, for example, charges a license fee of $12.50 for all passenger cars of 24 horsepower or less, $20 for cars from 25 to 30 horsepower, $30 for cars from 30 to 35 horsepower, and $40 for all cars of 35 horsepower and over. This schedule follows the grouping principle. Another type of horsepower schedule is found in New Jersey, where owners are charged a license fee of 40 cents per horsepower for cars of 29 horsepower or less and 50 cents for cars of horsepower greater than 29. This is a more flexible schedule which attempts to obviate the sudden increases in fees that are bound to occur where the fees are raised appreciably at the end of a group classification. It is intended to bring about more uniform charges.

Classification of automobiles according to the weight of the vehicle is another basis of determining license fees, which in 1923 was followed by seven States. South Dakota, for example, charges a license fee of $13 for automobiles of less than 2,000 pounds, $17 for cars from 2,000 to 3,000 pounds, $20 for cars from 3,000 to 4,000 pounds, and $35 for cars of 4,000 pounds and over. Other States charge a certain amount per unit of weight. New York, for instance, charges 50 cents per 100 pounds for cars weighing 3,500 pounds and less and 75 cents per 100 pounds for cars weighing over 3,500 pounds. This modification is made for the same reason that some of the States classifying according to horsepower charge definite rates per horsepower.

Six of the States classify automobiles for licensing purposes according to the gross weight including vehicle and load. Delaware's schedule of fees, according to which a charge of $2 is made for each 500 pounds of gross weight, is an example of this method. The gross weight is ascertained by adding to the weight of the car an estimated weight of 125 pounds for each person applied to the rated passenger capacity.

There are only two States, Colorado and Oklahoma, which use the value or cost of the passenger car as the basis for licensing charges. The application of such a principle is obviously somewhat complicated and can lead to certain dissatisfaction. Colorado charges one-half of 1 per cent of the manufacturer's price of the vehicle and allows a 30 per cent reduction after the fifth year of use and 50 per cent reduction after the eighth year of use. The Oklahoma scale is $10 for the first $500 value and 75 cents per $100 value in excess of $500. The use-of-the-road factor is in these cases evidently subordinated to considerations of the owner's ability to pay. The taxation principle is more closely adhered to than the principle of privilege.

Only one State, Connecticut, bases its schedules of license fees upon the displacement of the engine pistons, the charge being 8 cents per cubic inch. This is in fact another species of classification and charging according to horsepower.

Four States have in force and effect schedules of license fees based upon a combination of horsepower and gross weight and four States use a combination of horsepower and the weight of the vehicle only. The West Virginia schedule is an example of the former class, the rate being 30 cents per horsepower plus 30 cents for each 100 pounds gross weight (passengers estimated at 125 pounds each). Maine, on the other hand, charges 25 cents per horsepower and 25 cents per 100 pounds of the weight of the vehicle.

There are two States which base their license fees upon a combination of weight and value. Iowa, for example, has a schedule which provides for a charge of 1 per cent of the value plus 40 cents per 100 pounds. The executive council of the State has to fix every year the basic value and weight for computing fees. This combination is another example of the effort made by the legislature to give recognition to both the taxation and the privilege principles.

One State, North Dakota, employs a three-fold basis for license fees—value, net weight, and horsepower. For the first registration the fee is computed on one-half of 1 per cent of the value, 20 cents per 100 pounds, and 10 cents per horsepower. A 10 per cent reduction is made for the second registration, 25 per cent reduction for the third registration, and 40 per cent reduction for all other registrations. A $5 minimum fee is provided for in all cases.

DEVELOPMENT OF SPECIAL FEES FOR MOTOR TRUCKS

Specific license fees for motor trucks, generally spoken of as commercial cars, were not provided for in the early days when legislatures passed their first motor vehicle licensing acts because the motor truck had not yet made its appearance. The change in the type of fees applied to motor trucks is shown in the following tabulation, where the bases for determining the license fees for motor trucks are set forth for the years 1914, 1921, and 1924:

Methods of licensing motor trucks or commercial cars by the various States

Basis of license fee	Number of States [1]		
	1914	1921	1924
Capacity	2	25	25
Horsepower	17	2	
Net weight		2	2
Gross weight	1	7	10
Value		1	
Tire width		1	1
Weight of chassis		1	
Flat rate per vehicle	25	2	
Horsepower plus gross weight		2	2
Horsepower plus net weight		2	1
Horsepower plus capacity		1	2
Horsepower plus flat rate	1		
Weight plus horsepower plus value plus capacity		1	
Net weight plus capacity		1	
Value plus capacity			1
Flat rate plus capacity			1
Flat rate plus net weight			1
Value plus net weight plus horsepower			1
Horsepower plus weight plus capacity			1
No license law	2		
Total	48	48	48

[1] Sources same as those for passenger cars.

In 1914, 46 States charged fees for the licensing of motor vehicles and only 11 of these (Indiana, Maine, Maryland, Massachusetts, Oregon, Rhode Island, Connecticut, New Hampshire, New Jersey, New York, and Pennsylvania) made any special provision for motor trucks. The remainder of the States applied to motor trucks the same schedule of fees which was paid by passenger cars. Most of these charged flat rates or rates based upon the horsepower of the vehicle.

Those States which attempted to place motor trucks in a separate class and charge fees different from those assessed against passenger cars did so on two directly opposite theories. Seven of them (Indiana, Maine, Maryland, Massachusetts, New York, Oregon, and Rhode Island), although they were charging for passenger cars fees based upon horsepower, deliberately put motor trucks into a separate class and charged them flat fees which in several instances were less than the regular rates applied to passenger cars of similar horsepower. Maryland, for example, charged $5 for passenger cars of 10 horsepower and less, $10 for cars of 11–20 horsepower, $15 for cars of 21–30 horsepower, $20 for cars of 31–40 horsepower, and $25 for cars of over 40 horsepower, whereas all motor trucks were charged only $3. Likewise Massachusetts charged a license fee of only $5 for motor trucks of all sizes and capacities and at the same time had in effect a scale of fees ranging from $5 to $25 depending upon horsepower for passenger automobiles. This deviation in favor of the motor truck can be explained in two ways. First, the passenger car at that time was still regarded by many as a luxury and therefore capable of being heavily taxed, while the motor truck, it was held, was useful and, therefore, should be relieved of any undue burden. The effect of the motor truck upon the highways was not considered because its use was still rather limited. A second reason which may have been in the minds of those who felt that motor trucks should not pay as high a fee as passenger automobiles was that the motor trucks which were in use at that time were largely engaged in intracity operation; few were seen on the rural highways. These more or less nominal fees were charged rather on the police-power theory than on the theory that revenues should be derived from them for highway purposes.

Four States established special scales of fees for motor trucks. Connecticut and New Hampshire graded the fees according to the capacity of the truck, Pennsylvania charged according to gross weight, and New Jersey applied the regular fees based upon horsepower plus a fee of $10 on trucks weighing over 4,000 pounds. These States were the first to come to the conclusion that motor trucks on account of their weight damaged the highways more than passenger cars and should therefore be charged higher fees. By 1921 the States had developed 13 different methods of charging license fees for motor trucks and 1924 showed a number of modifications of these methods, all of which are intended to give the States revenues in proportion to the use which the trucks make of the highways and the wear and tear caused by them. Twenty-five of the States now classify the trucks according to carrying capacity alone and 10 base their schedules of fees on the gross weight of the truck and rated carrying capacity. In essence the theory underlying all of the various systems of classification is that the larger and the heavier the truck the higher should be the license fee. A number of States provide for differentials in fees based upon the fact as to whether pneumatic or solid tires are used. It is being recognized that if the license fee is to bear any relationship to the road damage caused by the truck the type of tires used must be taken into consideration.

VARIATION IN MOTOR TRUCK FEES GREATER THAN IN PASSENGER CAR FEES

The variation in the fees for a motor truck of any particular capacity in the various States is much greater than it is in the case of passenger automobiles. To show these differentials a comparison of license fees has been made for several different sizes of trucks, indicating the fees of the various States for 1914, 1921, and 1924. Three types of trucks are used in making these comparisons, involving the following assumptions: 1½-ton truck, solid tires, 23 horsepower, price $2,700, 4,900 pounds unloaded; 3½-ton truck, solid tires, 31 horsepower, price $4,800, 8,800 pounds unloaded; 5-ton truck, solid tires, 34 horsepower, price $5,800, 10,700 pounds unloaded. The averages of fees charged by all States for these assumed vehicles in the three years 1914, 1921, and 1924 are as follows:

Average license fees for 1½-ton, 3½-ton, and 5-ton trucks

Capacity of truck	1914	1921	1924
1½ tons	$6.43	$27.55	$31.15
3½ tons	8.36	64.05	85.75
5 tons	8.80	96.52	139.39

In 1914 motor truck fees were low in nearly all States, most of them, as already mentioned, applying to such vehicles the regular fee charged for passenger cars. By 1924 the average fees paid for a 1½-ton truck were 485 per cent of the 1914 fees; the fees of a 3½-ton truck were over 1,000 per cent of the 1914 fees; and a 5-ton truck fee had increased nearly 1,600 per cent. This greater increase in the fees paid by the larger trucks is wholly due to the fact that motor trucks had in the interim been classified in such a manner that the larger and the heavier types of trucks were placed into groups or classes to which were applied the higher fees.

The number of States charging various average fees for these three types of motor trucks in the three years are shown in the following table:

Number of States charging various average license fees for three sizes of motor trucks, 1914, 1921, and 1924

Average fees	1½-ton			3½-ton			5-ton		
	1914	1921	1924	1914	1921	1924	1914	1921	1924
Less than $10	33	4		29	2		29	2	
$10–$19	12	7	8	12	3		11	3	
$20–$29	1	17	20	4	3	2	4	2	1
$30–$39		9	8	1	4	3	2	1	
$40–$49		7	4		6	7		3	4
$50–$59		3	5		6	4		5	6
$60–$69		1	1		5	4		3	2
$70–$79			1		2	5		4	4
$80–$89					3	3		3	3
$90–$99			1		4	2		1	
$100–$109					8	8		3	2
$110–$119						1		1	1
$120–$129					1	2		2	3
$130–$139						1			
$140–$149						3		3	4
$150–$159								4	7
$160–$169									
$170–$179						1		1	
$180–$189								2	3
$190–$199									
$200–$209					1	1		4	3
$210–$219									
$220–$229						1			
$230–$239									1
$240–$249									
$250–$259									
$260–$269									
$270–$279									
$280–$289									
$290–$299									
$300–$309								1	3
$310–$319									
$320–$329									
$330–$339									
$340–$349									
$350–$359									
$360–$369									
$370–$379									
$380–$389									
$390–$399									
$400–$409									1
Total	46	48	48	46	48	48	46	48	48

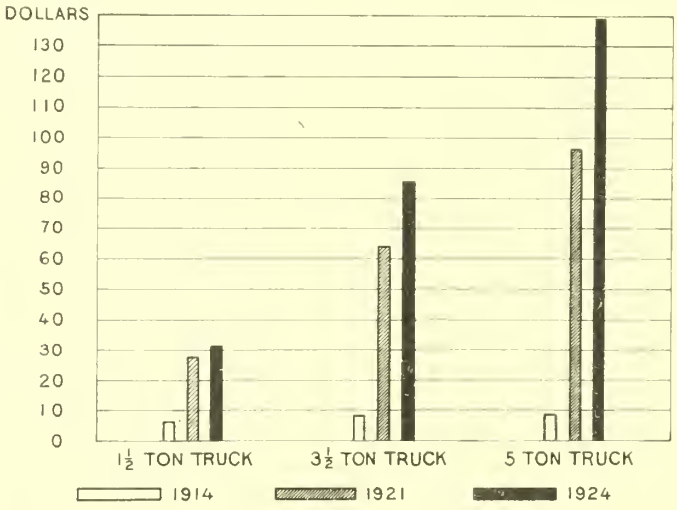
The highest license fee charged for any truck in 1914 was $34 for a 5-ton truck; two States charged that fee. In that year a majority of the States charged less than $10. While the fees had been advanced very much by 1921 the fees for 1924 were still higher. In 1924 only eight States charged less than $20, and that was for the 1½-ton truck. The tabulation shows how great the variation is—a 5-ton truck can be licensed in one State for from $20 to $29; in another State it has to pay between $400 and $409. In about half the States the fee for a 5-ton truck is less than $120; in the rest of the States it ranges from $120 to $409. It is evident that there is a tendency to fix fees at constantly higher levels for the motor trucks and particularly those of large capacities.

The differentials in the rates of passenger automobiles are much less. The lowest rate for a five-passenger car of 25 horsepower in 1924 is $3 and the highest fee charged by any of the States on a similar car is $46.20, which is unusually high. In 25 States in which complete information could be obtained the average license fee for passenger automobiles in 1923 was $10.78 and $18.23 for motor trucks.

There is no indication of a uniform differential between the average fees charged for passenger cars and for motor trucks. Only about half of the States have compiled their motor vehicle registration data in such manner that a segregation can be made of the revenues collected from passenger cars and from motor trucks. The following tabulation shows the States making this division and the average fees collected in 1923 from each class of motor vehicles, also the percentage relation which the motor truck fees bear to the passenger car fees:

Comparison of averages fees for passenger cars and motor trucks, 1923

State	Average passenger car fees	Average motor truck fees	Ratio of motor truck fees to passenger car fees
			Per cent
Arkansas	$12.00	$17.00	142
California	8.59	18.70	218
Colorado	5.12	11.57	226
Connecticut	15.47	32.82	212
Delaware	11.65	20.57	176
Georgia	14.12	16.86	119
Illinois	8.39	14.91	178
Indiana	5.20	10.84	208
Louisiana	15.52	16.30	105
Massachusetts	10.58	15.21	144
Michigan	12.38	17.03	137
Minnesota	15.55	19.67	126
Montana	9.24	11.12	120
Nebraska	10.60	18.70	177
Nevada	8.75	15.00	171
New Jersey	9.29	27.02	291
New Mexico	8.68	12.00	138
New York	12.14	26.45	218
Rhode Island	12.00	19.76	164
South Carolina	6.29	13.27	211
Vermont	13.66	24.85	183
Virginia	12.89	15.61	121
Washington	12.47	22.72	182
Wisconsin	10.00	18.11	181
Wyoming	8.90	19.66	221
Average	10.78	18.23	169

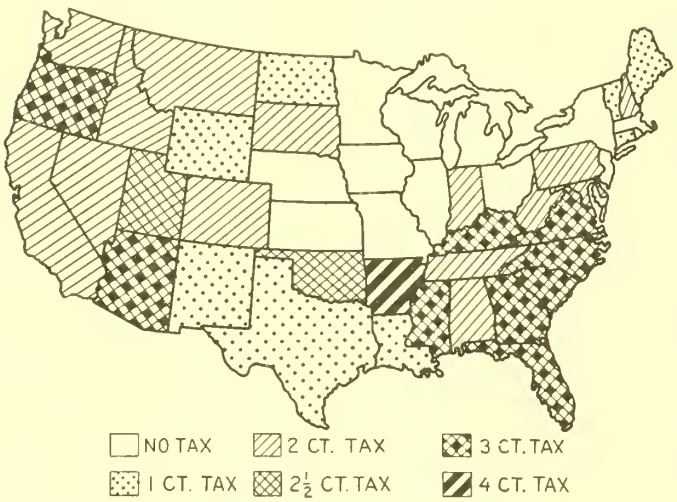


Average license fees for 1½, 3½, and 5 ton trucks, 1914, 1921, and 1924

In these States the average passenger-car fees range from $15.55 in Minnesota to $5.12 in Colorado; the motor truck fees from $32.82 in Connecticut to $10.84 in Indiana. The rates for the two classes of vehicles show the greatest difference in New Jersey, where the average motor-truck fee is 191 per cent higher than the average passenger-car fee. The smallest differential appears in Louisiana, where the average motor-truck fee is only 5 per cent above the average fee received from passenger cars. In only eight of the States are the average motor-truck fees more than double the fees of passenger cars. The average motor-truck fees are 69 per cent above the average fees charged for passenger cars in these States. If we apply the 1924 scales of license fees existing in the 48 States to an average car of 25 horsepower we get an average fee of $15.38.

USE OF MOTOR-VEHICLE REVENUE BY STATES AND COUNTIES

Although all but two of the States were collecting fees from the licensing of motor vehicles in 1914 only 19 of them [1] turned over the funds obtained from this source to their respective State highway departments for construction and maintenance purposes. In one of these States—Maryland—the law provided that one-fifth of the revenue collected should be used within the city of Baltimore and the balance for the maintenance of State roads. The motor-vehicle laws of nine States [2] provided that the revenues derived from motor vehicles should be paid to the counties at stated times for use on the highways maintained by them. In a few cases it was provided that the registration and administration expenses should be first deducted before turning over the funds to the counties. In every case except one the money which the counties received was the money paid in fees by the residents of the counties. The Georgia law employed as the basis of apportionment the provision that the funds be apportioned among the counties in accordance with the road mileage of the respective counties. In those States where the counties received practically all of the motor-vehicle fees there were at the time no State



Rates of gasoline tax levied, July, 1924

highway departments which could assume control and supervision of the expenditure of funds for highway purposes. Road construction and maintenance had always been matters which were left largely to local authorities. From time to time funds were appropriated by the legislature and turned over to local bodies in the shape of aid for certain extraordinary highway construction expenditures. This was usually done where expensive bridges were built on highways which carried long-distance traffic and where highways were built which were regarded as trunk roads and which partook of the character of State roads because of the nature of the traffic.

Ten States [3] followed the practice of dividing these funds between the State and the counties; in most cases the money accruing to the State was turned over to its highway department and the money turned over to the counties was to be used by them for local road work. In Arkansas the money was equally divided between the State and the counties; the State's portion of these license revenues was given to the State highway department and the share going to the counties was transferred to the school funds of the several counties. At the same time the Ohio law provided that one-third of the license fees should be turned over to the counties to be used for highway purposes and two-thirds be retained by the State and placed in the general fund.

Six [4] States kept all the motor-vehicle revenues in the general fund, making no apportionment and no direct provision for highway use. Alabama divided these license revenues between the general fund of the State and the general funds of the local governmental units. In Texas a fee of 50 cents was charged for the registration of a motor vehicle, and funds resulting therefrom were retained by the county clerks, who were the licensing officers specified by law.

In 1923 of the $188,613,074 collected by the States as motor-vehicle license revenues ($357,918 collected by the District of Columbia not included) $153,226,636, or 81 per cent, was used for highways by and under the direction and supervision of State highway departments. In only 13 States were any substantial amounts apportioned among the counties. The laws in some of the States provided for the motor-vehicle registration and licensing expenses to be paid out of these specific revenues; in other States the costs connected with this work are charged to the general funds. The 19 per cent which was not used by the State highway departments includes in addition to the funds turned over to local units the expenses of registering cars and the issuing of licenses and other administrative expenses of similar character.

At this time every State has come to recognize the relationship between the licensing of automobiles and the procuring of funds for highway purposes. Among those States where the fees have been low in the past steps are taken to augment the highway funds through higher registration fees.

THE GASOLINE TAX AS A PRODUCER OF HIGHWAY REVENUES

At about the time when most of the States had succeeded in abolishing the old turnpikes and toll roads by purchasing them at agreed prices or through condemnation proceedings and had declared them "free" highways a movement started which in essence was an extension of the toll principle to all the highways of a State brought about through the enactment of laws providing for a tax upon the sale of gasoline. In 1919 two States—Oregon and Colorado—passed a gasoline tax law; this year, 1924, there are 35 States and the District of Columbia which are collecting this kind of tax. Seventy-three per cent of the States, in other words, have in effect a gasoline tax; and these States have a rural road mileage of 1,954,886 miles comprising 66.5 per cent of the total rural highway mileage of the country.

[1] Arizona, Connecticut, Georgia, Illinois, Kentucky, Maine, Maryland, Massachusetts, Michigan, Missouri, New Jersey, New Mexico, New York, Oklahoma, Pennsylvania, Rhode Island, Utah, Vermont, and Virginia.

[2] Florida, Indiana, Kansas, Nebraska, Nevada, New Hampshire, North Dakota, Oregon, and Wyoming.

[3] Arkansas, California, Colorado, Idaho, Iowa, Montana, North Carolina, Ohio, South Dakota, and Wisconsin.

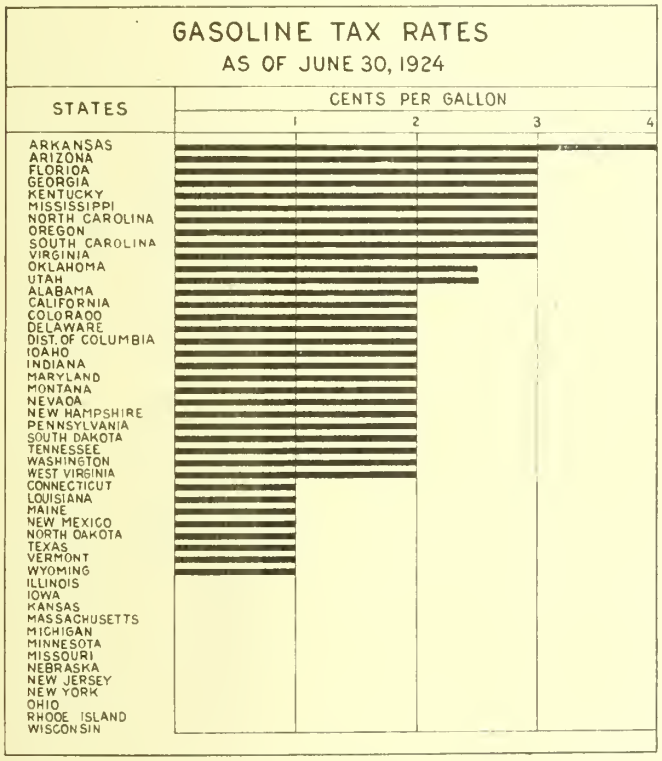
[4] Delaware, Minnesota, South Carolina, Tennessee, Washington, and West Virginia.

The rates charged by these States range from 1 cent to 4 cents per gallon. The tendency has been for more States to enact gasoline tax laws and for States having the law to increase the rate of the tax. In the following tabulation is presented a list of the States levying this tax and the present rates:

States levying gasoline taxes classified according to rate [1]

1-cent tax	2-cent tax	2½-cent tax	3-cent tax	4-cent tax
Connecticut.	Alabama.	Oklahoma.	Arizona.	Arkansas.
Louisiana.	California.	Utah.	Florida.	
Maine.	Colorado.		Georgia.	
New Mexico.	Delaware.		Kentucky.	
North Dakota.	Idaho.		Mississippi.	
Texas.	Indiana.		North Carolina.	
Vermont.	Maryland.		Oregon.	
Wyoming.	Montana.		South Carolina.	
	Nevada.		Virginia.	
	New Hampshire.			
	Pennsylvania.			
	South Dakota.			
	Tennessee.			
	Washington.			
	West Virginia.			

[1] Massachusetts passed a 2-cent tax law; it is held in abeyance and referred to the people for approval at the November, 1924, election. District of Columbia charges a 2-cent tax under act of Congress.



Of the 35 States, 15 levy a 2-cent tax, 8 a 1-cent tax, 9 a 3-cent tax, 2 a 2½-cent tax, and 1 State is charging a tax of 4 cents a gallon. The States which are not collecting this form of tax are all in the east north central and west north central sections, with the exception of Massachusetts, Rhode Island, New York, and New Jersey. It is evident that there is a solid South and a solid West in favor of this type of tax. During the year 1923 these States received $36,813,939 in gasoline taxes.

LESS THAN 60 PER CENT OF GASOLINE TAXES EXPENDED UNDER STATE SUPERVISION

The disposition of the revenues derived from the gasoline tax differs from that of the motor vehicle license fees. In 1923 about 81 per cent of the motor vehicle license revenue was applicable to highway work by or under the supervision of State highway departments, whereas only 58.5 per cent of the total gross receipts accruing from the gasoline tax was disposed of in this manner. In the following table the disposition of the gasoline tax receipts of the States collecting this tax is compared with the disposition which these same States made of the motor vehicle license fees in 1923. In each case the percentage for use by the State highway departments is given.

Percentage of motor vehicle license fees and gasoline taxes applicable to highway work under the supervision of the State highway departments, 1923

State	Motor vehicle fees	Gasoline taxes	State	Motor vehicle fees	Gasoline taxes
	Per cent	*Per cent*		*Per cent*	*Per cent*
Alabama	78	0	New Mexico	95	95
Arizona	100	25	North Carolina	99	99
Arkansas	30	24	North Dakota	99	0
California	46	50	Oklahoma	90	100
Colorado	48	47	Oregon	72	97
Connecticut	100	100	Pennsylvania	100	0
Delaware	100	100	South Carolina	80	27
Florida	71	70	South Dakota	93	91
Georgia	97	16	Tennessee	99	99
Idaho	25	100	Texas	44	75
Indiana	95	86	Utah	100	26
Kentucky	100	100	Vermont	92	100
Louisiana	100	100	Virginia	100	67
Maine	89	99	Washington	96	100
Maryland	90	100	West Virginia	100	100
Mississippi	54	40	Wyoming	100	100
Montana	10	17			
Nevada	94	52	Average	81	58.5
New Hampshire	93	99			

Taking these States as a whole it is evident that the State highway departments expend or control the expenditure of a much larger portion of the motor vehicle license funds than they do of the gasoline tax funds. In the case of the gaoline tax there is a much greater tendency for the State to share these newly discovered revenues with the counties and to divert them for other purposes than in the case of the motor vehicle license fees. There are now 17 States which provide in their gasoline tax laws that the full proceeds of this tax shall be used by the respective State highway departments or under their supervision. These States are Arkansas,[5] Connecticut, Delaware, Idaho, Kentucky, Louisiana, Maryland, New Hampshire, North Carolina, Oregon, South Dakota, Tennessee, Utah, Vermont, Washington, West Virginia, and Wyoming. In some of the States the costs of collecting the tax are deducted from the receipts and the net proceeds are made available for State highway purposes; in other States the total gross receipts are transferred to the highway funds, in which cases the collection costs are met out of other funds. In any event they are usually small in amount and the work is done in many instances by the regular State officials and employees, making it unnecessary to incur any extra expenses.

Ten of the States (California, Colorado, Florida, Indiana, Maine, Mississippi, Nevada, Oklahoma, Pennsylvania, and Virginia) make a division of the gasoline tax revenues between the respective State highway departments and the county highway organizations. In some of the States there is an equal division of the tax in a few States two-thirds of the tax goes to the State and one-third to the counties; in others the division is made on the basis of 60 per cent to the counties and 40 per cent to the State. In one State three-fourths of the tax is turned over to the counties and one-fourth is retained by the State; in another State the reverse of this division obtains.

[5] Effective Jan. 1, 1924.

One State—North Dakota—places the whole proceeds of the tax in the general fund of the State, no part of it being devoted directly to highway purposes. Alabama makes a disposition of the gasoline tax revenues by putting one-half into the general fund of the State and by letting the counties use the other half for highway purposes. In Texas, three-fourths of the tax is used by the State highway department and the other one-fourth is credited to the State school funds. South Carolina, Montana, and Georgia make a threefold division of the proceeds—to the State highway department, the county highway organizations, and the State general fund. South Carolina makes an equal apportionment among these three claimants; Montana credits the State highway funds with 20 per cent and the county highway funds and the State general funds with 40 per cent each. In Geogia one-third of the gasoline tax goes to the State highway fund and one-third to the county and one-third is credited to a special State fund to meet the State's obligations in connection with bonds which the State issued for the construction of a railroad.

In Arizona the gasoline tax of 3 cents is collected by the Secretary of State and apportioned by him as follows: 50 per cent of the tax from each county directly to the county in which it is collected and 50 per cent to the State treasurer. The revenue is then used as follows: 25 per cent for State road work and 75 per cent for county road work. In New Mexico $15,000 of the tax collected is credited to the State fish hatchery fund and the balance to the State road funds.

It is noted that in a number of cases a portion of the gasoline tax receipts is credited to the general fund of the State. This is done in some instances because the State has issued highway bonds, and this gasoline tax money is intended to take care of those obligations.

AVERAGE ANNUAL GAS CONSUMPTION IN EIGHT STATES 450 GALLONS

On account of the fact that so many changes in the tax rate took place in these States during the year 1923, it is impossible to ascertain the annual gasoline consumption per vehicle except for those States where the rate remained the same throughout the whole year. In the following table the gross receipts, number of motor vehicles registered, the receipts per motor vehicle, and the average consumption per vehicle are set forth for the eight States in which the same rate prevailed throughout the year:

Gross receipts derived from gasoline taxes and average receipts and gasoline consumption per vehicle, 1923

State	Gross receipts	Number of motor vehicles	Gross receipts per vehicle	Gasoline consumption per vehicle [1]
				Gallons
Connecticut	$880,222.70	181,748	$4.84	484
Kentucky	680,435.30	198,377	3.43	343
Louisiana	754,437.85	136,622	5.52	552
Maryland	688,304.02	169,351	4.06	406
Mississippi	467,855.53	104,286	4.49	449
New Mexico	165,000.00	32,032	5.15	515
North Dakota	461,081.71	109,266	4.22	422
Washington	1,225,149.66	258,264	4.74	474
Total	5,322,486.77	1,189,946	4.50	450

[1] The tax rate in all of these States was 1 per cent per gallon.

At a rate of 1 cent per gallon this tax yielded in these eight States $4.50 per motor vehicle, indicating that the annual consumption was, on the average, 450 gallons. Using this average as a basis for computation, it is easy to estimate the average revenues per vehicle which can be derived where higher rates per gallon are collected.

If we assume 6,000 miles as the average annual mileage per vehicle (this is an average of 13.3 miles per gallon of gasoline) we arrive at the following toll charges per vehicle-mile for the use of the highways:

Gasoline tax	Toll per vehicle-mile
Cents per gallon	*Cents*
1	0.075
2	.150
3	.225
4	.300
5	.375

According to these estimates a 1-cent gasoline tax means that the motor-vehicle operator would be called upon to pay a toll charge of 10 cents for a trip from Washington to Philadelphia. The trip over the Lincoln Highway from New York to San Francisco would be taxed $2.50.

It is interesting to compare these tax rates with the toll charges which motorists formerly had to pay on toll roads and turnpikes. On six different turnpikes in Virginia and Maryland tolls amounting to $5.05 were charged for a total distance of 187.5 miles [6] which is equivalent to 2.7 cents a mile. If a State attempted to charge this same rate for the use of the public highways by automobiles it would have to establish a gasoline tax at 36 cents per gallon, assuming the same consumption per mile as in the foregoing analysis. Compared with these former toll road charges it will be seen that the gasoline taxes now levied are extremely moderate.

THE APPLICATION OF THE GASOLINE TAX

The States have not gone very far in laying down precise definitions of the products to be taxed in the statutes passed thus far. The intent of the legislation generally is to tax all fuels which are used in the operation of motor vehicles propelled by internal-combustion engines. In some States the law names gasoline and no other kind of fuel, in others kerosene is included, and in some it is specifically exempted from the provisions of the tax. Some of the laws contain elaborate provisions exempting motor fuels which are used in farm tractors, motor boats, and airplanes, the purpose being to tax only the fuel used by vehicles which are operated over the public highways. Such an application of the law is in accordance with the toll-road principle. In certain States all the gasoline sold within the State is subject to the tax irrespective of the use to which it is put. Under such conditions the clothes-cleaning establishment using a certain amount of gasoline, as well as the motor boat operator and the aviator, have to make their contribution to the maintenance of the highway in proportion to the amount of gasoline they consume. Where exemptions are made based upon the use to which gasoline is put it is the general practice to compel the payment of the tax when the purchase is made and allow the purchaser to file a claim for a refund based upon an affidavit that the gasoline was not used as fuel for a highway vehicle.

[6] American Automobile Blue Book, vol. 3, 1919.

License fees and gasoline taxes per vehicle-mile based on average travel of 6,000 miles, 1923

State	Average license fee per vehicle-mile	Average gasoline tax per vehicle-mile[1]	Combined taxes per vehicle-mile
	Cents	*Cents*	*Cents*
Alabama	0.203	0.149	0.352
Arizona	.095	.160	.255
Arkansas	.211	.179	.390
California	.160	.038	.198
Colorado	.099	.075	.174
Connecticut	.397	.081	.478
Delaware	.287	.049	.336
District of Columbia	.080		.080
Florida	.215	.180	.395
Georgia	.207	.144	.351
Idaho	.244	.106	.350
Illinois	.167		.167
Indiana	.105	.083	.188
Iowa	.258		.258
Kansas	.152		.152
Kentucky	.225	.057	.282
Louisiana	.267	.092	.359
Maine	.255	.044	.299
Maryland	.348	.068	.416
Massachusetts	.242		.242
Michigan	.239		.239
Minnesota	.272		.272
Mississippi	.172	.075	.247
Missouri	.140		.140
Montana	.165	.100	.265
Nebraska	.195		.195
Nevada	.163	.123	.286
New Hampshire	.439	.046	.485
New Jersey	.296		.296
New Mexico	.153	.086	.239
New York	.274		.274
North Carolina	.252	.196	.448
North Dakota	.116	.070	.186
Ohio	.151		.151
Oklahoma	.175	.033	.208
Oregon	.408	.196	.604
Pennsylvania	.253	.088	.341
Rhode Island	.281		.281
South Carolina	.118	.198	.316
South Dakota	.143	.079	.222
Tennessee	.197	.078	.275
Texas	.132	.029	.161
Utah	.120	.113	.233
Vermont	.296	.053	.349
Virginia	.244	.118	.362
Washington	.252	.079	.331
West Virginia	.276	.039	.315
Wisconsin	.181		.181
Wyoming	.173	.059	.232
Average	.208	.041	.249

Toll per vehicle-mile on 187.5 miles of turnpike in Maryland and Virginia, 2.7 cents.

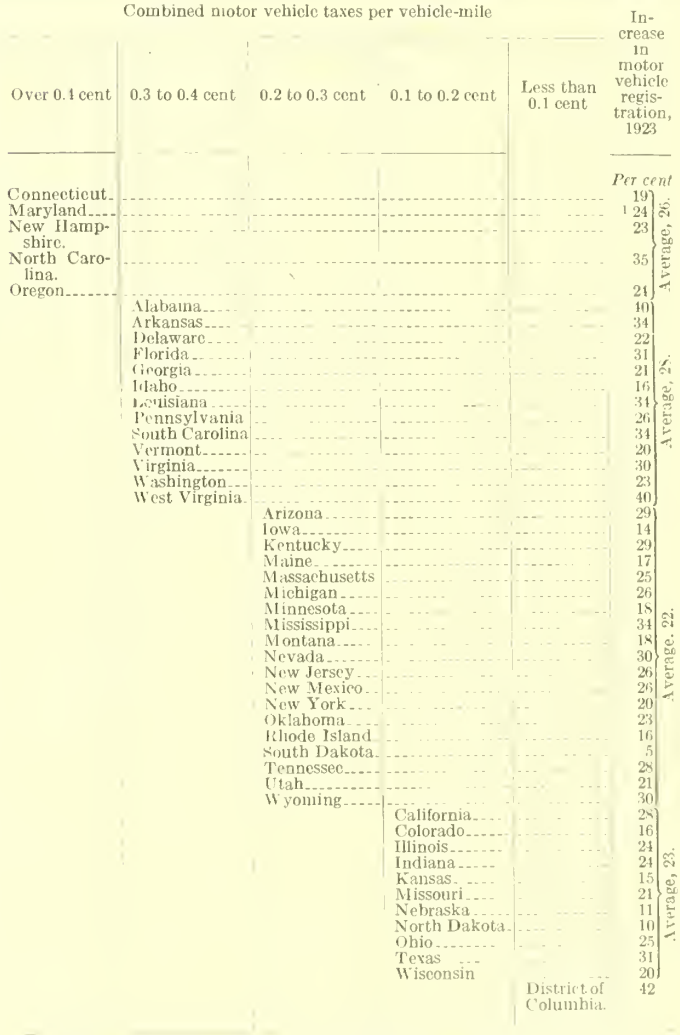
[1] A number of the figures in this column are based on collections of gasoline taxes for less than one year. (See PUBLIC ROADS, vol. 5, No. 2, April, 1924, p. 17.)

The cost of collecting the tax is very small. Few taxes are collected at so moderate a cost. In most States collection is exacted from the wholesale distributor and precaution is taken through periodic reports that the tax is not collected twice. According to the apparent trend it will not be long before all States will be employing the gasoline tax to secure revenues for highway purposes. Three States—Wisconsin, Michigan, and Iowa—in addition to the 35 which now use the tax, have pushed gasoline tax bills through the legislatures, but in each case they were vetoed by the governors. In each case the veto was based on the ground that the tax was a sales tax and for that reason undesirable, or on some fault in the proposed method of distributing the revenue.

In only a few States where the gasoline tax has been introduced has a revision of the motor vehicle license fees been made, relieving the motor vehicle operator in any way of a part of those charges. California reduced the license fees to a flat rate of $3 for all types of motor vehicles. In Maryland, when the gasoline tax was raised from 1 cent to 2 cents a gallon, the legislature took the definite position that additional revenues were not demanded through the imposition of the larger gasoline tax and the highway authorities were requested to make a readjustment in the motor-vehicle fees to offset the increased gasoline tax. A reduction in fees was made which it was estimated would about equal the average amount each class of motor vehicle would have to pay in additional gasoline taxes.

It would appear so far as can now be learned from an analysis of the development of the gasoline tax that it is going to be a permanent method of raising a part of the funds necessary for highway purposes. It is distinctly a tax for highway service, and, considering the character of the service rendered in return, none of the rates now charged is excessive. Even when the gasoline tax is coupled with the license fee for the motor vehicle the combined taxes considered as toll for the use of the roads are extremely moderate. Compared with the turnpike charges referred to above, for example, the charges per mile for license fees and gasoline taxes, assuming an average annual mileage of 6,000 miles, seem rather insignificant.

Classification of States according to combined motor vehicle taxes per vehicle-mile in relation to increase in motor vehicle registrations, 1923



[1] Nonresident registrations included in both years in making this computation.

(Continued on page 24)

IMPACT TESTS ON HIGHWAY BRIDGES

Progress Report of Cooperative Research Conducted at Ames, Iowa

IMPACT stresses in highway bridges subjected to motor-truck loads are given in the progress report of the highway bridge impact tests recently made to the cooperating agencies, the Iowa State Highway Commission, the Engineering Experiment Station of Iowa State College, and the United States Bureau of Public Roads, by Almon H. Fuller, consulting bridge engineer of the Iowa commission.

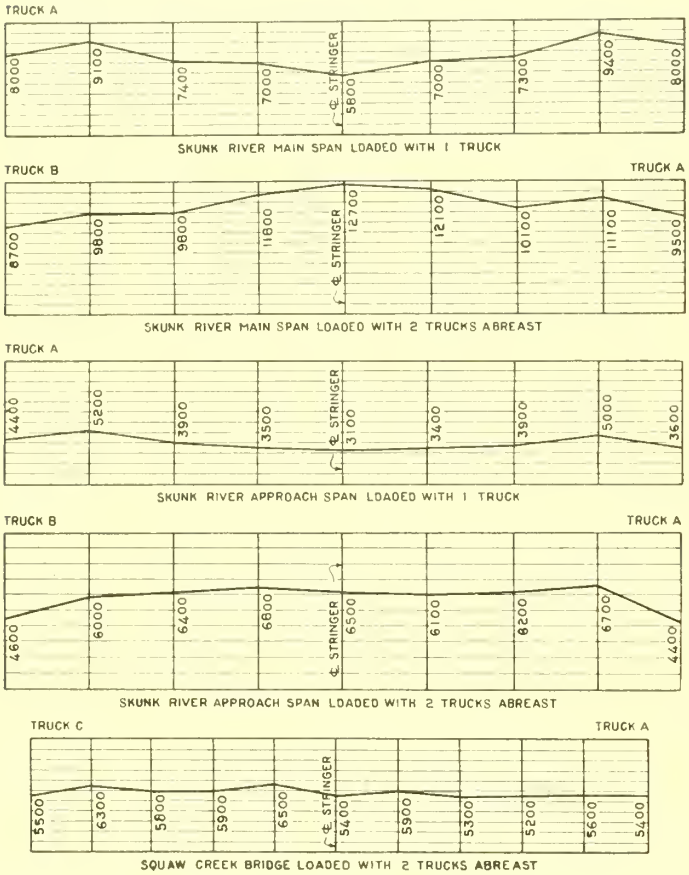


FIG. 1.—Distribution of stresses in stringers, maximum stress in each stringer due to various positions of loads

The tests were begun in 1922 and a preliminary report of the work of that year was published as Bulletin No. 63 by the Engineering Experiment Station. The recent report covers the continuation of the work in 1923. The greater part of the season's work was done on the 8-panel, 150-foot main span, and the 32-foot approach span over the Skunk River on the Lincoln Highway near Ames, Iowa, the same structures which were used in the 1922 work. A few days' work was done on each of the two other structures, one a 70-foot through plate girder bridge with an 8-inch concrete floor supported on steel stringers, located on the Lincoln Highway over Squaw Creek in the city of Ames. The other was a 5-panel, 40-foot, riveted steel, pony truss bridge with 8-inch reinforced concrete slab resting directly on floor beams, located near Roland, Iowa.

The 1923 work included observations of static load and impact stresses on all the bridges. Three conditions of impact were observed—one with the trucks operated over the clean bridge floor, one in which the trucks were driven over a 1 by 2 inch obstruction, and one in which they were driven over a 2 by 4 inch obstruction.

Probable percentages to be added to static load stresses to allow for impact are given as follows:

For stringers: Clean floor, 12 per cent; 1-inch obstruction, 40 per cent; 2-inch obstruction, 80 per cent.

For floor beams: Clean floor, 10 per cent; 1-inch obstruction, 40 per cent; 2-inch obstruction, 57 per cent.

For hip verticals: Clean floor, 25 per cent; 1-inch obstruction, 90 per cent; 2-inch obstruction, 150 per cent.

Similar percentages for main truss members are also given in the detailed tables accompanying the report. They are omitted above not because the results do not command confidence but because they are few in number and because the unit stresses are so small that they are not representative of fully loaded structures.

DISTRIBUTION OF STATIC STRESS IN STRINGERS

An extension of the static readings which were necessary as a base for determining the impact percentages gave the data for the distribution of stresses in stringers due to loads at rest. In the 1922 work the static readings were made with the trucks at rest. In 1923 they were made with the trucks operating at very slow speed. The hardly noticeable impact thus introduced was less important than the certainty that the correct position of the load was assured.

The maximum stresses in each stringer of three of the bridges under one and two trucks respectively are given in Figure 1. The stresses shown in the diagram are the maxima resulting from various positions of the truck across the width of the slab. Where two trucks were used they were placed abreast.

Two 3½-ton trucks loaded with gravel were used, the total loads averaging approximately 14 tons. As each truck was unloaded once on account of accident there were four distinct truck loads, designated A, B, C, and D. The axle weights for the four loads were as follows.

	Front axle load	Rear axle load	Total load
	Pounds	*Pounds*	*Pounds*
Truck A	5,910	23,410	29,320
Truck B	5,830	21,710	27,540
Truck C	5,800	22,200	28,000
Truck D	5,740	22,780	28,520

The distribution of static stress in the stringers under one truck is much the same as reported for the 1922 work in Bulletin No. 63 of the Engineering Experiment Station, Iowa State College. The 1923 work commands greater confidence because readings were taken on both flanges of every stringer. This was not done in all instances in 1922.

On a few of the stringers strain gauge readings were taken at various points on vertical sections as a means of locating the neutral axis. In each case the neutral axis was found to be above the center of the web, indicating a certain amount of T-beam action of the concrete floor. Besides observing the stresses they were computed under the assumption (1) that the load was supported entirely by the steel stringers as simple beams, and (2) that there was perfect T-beam action between the steel and the concrete.

Comparison of the observed and computed stresses shows that the observed stresses in all instances are so far below the stresses computed on the assumption that all load was carried by the stringers as simple beams as to leave no doubt that the T-beam action of the concrete floor or the restraining effect of end connections or both relieve the simple steel stringer of a considerable portion of the normal stress.

In two out of three cases the observed stresses were so near the ones computed on the assumption of perfect T-beam action and simple beams as to indicate that an excellent bond existed between the concrete floor and the steel stringers. The fact that the observed stresses while comparatively close to the computed ones, were always lower suggests a certain amount of restraint at the ends or partially fixed beams.

The observed position of the neutral axis of stringers in the Skunk River approach span (about 5 inches above the center of the web) compares so favorably with the computed position, assuming T-beam action, as to suggest that a complete bond existed. The observed position in stringers of the west panel of the main Skunk River span (about 1 inch above the center of the web) is sufficiently below the computed position (about 4 inches above the center of the web) to suggest that the bond was impaired on one or more of the stringers. As the neutral axis was determined experimentally for only one stringer, a more definite statement is not justified.

No strain gauge readings were taken on stringers of the Squaw Creek Bridge to determine the position of the neutral axis. The close comparison of observed computed stresses on the basis of perfect T-beam action, and the ratio of modulii of elasticity of 10 (from determinations on Skunk River approach span) suggest excellent bond.

COMPARISON OF OBSERVED AND COMPUTED STRESSES IN OTHER MEMBERS

Static load stresses were computed at all points at which they were observed and for identical loads. Close comparisons are not expected for three reasons:

1. Observed stresses were not taken at a sufficient number of points in the cross section of each member to represent the average distribution.

2. A high degree of precision can not be expected in observed stresses. It is believed that all of the averages are within 500 pounds per square inch and that many of them are within 200 pounds.

3. A high degree of precision in the computed stresses can not be expected because of the effect of the continuous steel and concrete floor. The results give strong indication that the floor relieves the lower chords of a considerable portion of their tension, raises the plane of bottom tension, and thereby reduces the effective depth of the truss and increases the compression in the upper chords. It is also likely that the web members are affected by the shear in the floor and that the general distributing power and continuity of the floor still further affects the stresses throughout the structure.

Tables 1, 2, and 3 show comparisons of observed and computed stresses in trusses, girders, and floor beams. The computations were made by the "ordinary" method and by "special" methods in which the effect of the floor was considered. A few irregularities exist which are not readily explained. When it is remembered, however, that comparatively little emphasis was placed on truss work in 1923, these comparisons seem reassuring rather than disturbing. The small intensity of the stresses suggests that greater loads are a necessity for satisfactory work on trusses.

TABLE 1.—*Comparison of observed and computed stresses on Skunk River Bridge*

[All trucks headed west]

Member	Load	Position of load	Observed unit stresses	Computed unit stresses	
				Ordinary method	Special method
			Pounds	*Pounds*	*Pounds*
Diagonal U1′–L2′	A	2 feet S. of N. curb	1,800	2,070	
	A and B	2 feet S. of N. curb	2,500	3,570	
Diagonal U1–L2	C and D	2 feet N. of S. curb	2,500	3,230	
	C	2 feet N. of S. curb	2,300	2,500	
	C and D	D–2 feet S. of N. curb	2,500	3,110	
Diagonal U2′–L3′	A	2 feet S. of N. curb			
	A and B	2 feet S. of N. curb	4,500	7,220	
			−1,800	−1,870	
Diagonal U3–L4	A and B	2 feet S. of N. curb	3,000	4,720	
			−1,400	−1,860	
Diagonal U3–L4	A and B	2 feet S. of N. curb	2,500	4,000	
			−2,000	−2,960	
Diagonal U4–L3′	A and B	2 feet S. of N. curb	2,500	2,250	
			−1,700	−2,860	
Diagonal U4–L3	A and B	2 feet S. of N. curb	2,900	2,880	
			−2,200	−2,430	
End post L0–U1	C and D	2 feet N. of S. curb	1,400	1,600	
	C and D	D–2 feet N. of S. curb			
		C–2 feet S. of N. curb	1,000	1,220	
Hip vertical U1–L1	A and B	A–2 feet N. of S. curb			
		B–2 feet S. of N. curb	3,450	3,540	
Hip vertical U1–L1	C	1 foot N. of S. curb	3,000	2,790	
	C	2 feet N. of S. curb	3,000	2,630	
Post U2′L–2′	A and B	2 feet S. of N. curb	1,700	2,180	
			−1,500	−1,230	
Floor beam L1	A and B	A–2 feet S. of N. curb			6,600
		B–2 feet N. of S. curb	7,000	9,270	

Figures preceded by a minus sign are the reverse stresses in the members.

TABLE 2.—*Comparison of observed and computed stresses on Roland 40-foot pony truss bridge*

[All trucks headed north]

Member	Load	Position of load		Observed unit stresses	Computed unit stresses	
		Rear wheels of A	Rear wheels of C		Ordinary method	Special method
				Pounds	*Pounds*	*Pounds*
L0–L1–L1–L2	A and C	L1	L1	1,820	6,230	2,280
	A and C	Midspan	Midspan	2,720	5,600	2,720
L2–L2′	C		Midspan	1,810	3,990	1,835
L0–U1	A and C	L1	L1	4,510	4,970	
U1–U2	A and C	L2	L2	4,620	5,680	6,270
U2–U2′	A and C	Midspan	Midspan	4,720	4,410	5,240
	C		Midspan	3,350	3,060	3,530
U1–L1	A and C	L1	L1	2,400	3,600	
U1–L2	A and C	L2	L2	5,620	8,750	
U2–L2	A and C	L2	L2	2,610	2,760	
	A and C	L3	L2	1,520	1,190	
U2′–L2	A and C	L2	L2	1,550	2,760	
Floor beam L1	A and C	L1	L1	5,710	16,650	8,800

TABLE 3. *Comparison of observed and computed stresses on Squaw Creek through girder bridge*

[All trucks headed west]

Member	Load	Position of load		Observed unit stresses	Computed unit stresses	
		Truck A	Truck C		Ordinary method	Special method
				Pounds	*Pounds*	*Pounds*
Top flange, north girder.	A and C	2 feet S. of N. curb.	11.5 feet S. of N. curb.	3,000	3,000	3,270
	A and C	2 feet S. of N. curb.	2 feet S. of N. curb.	3,000	3,510	3,800
Bottom flange, north girder.	A and C	2 feet S. of N. curb.	11.5 feet S. of N. curb.	2,500	3,000	2,300
	A and C	2 feet S. of N. curb.	2 feet S. of N. curb.	2,250	3,510	2,680
Floor beam J3.	A and C	9.5 feet S. of N. curb.	17 feet S. of N. curb.	4,270	8,770	6,160

MEASUREMENT OF IMPACT STRESSES

For the measurement of impact stresses the instruments used in the previous year's work which proved to be best adapted were retained. One new instrument, a six-element electrical, remote-reading and recording instrument developed by the United States Bureau of Standards was used for a few days. This instrument, a photogragh of which is shown in Figure 2, is decribed in United States Bureau of Standards Bulletin 247 under the title of "A New Electrical Telemeter" by Burton McCollum and O. S. Peters.

All the instruments were calibrated in the labratory for static loads on steel rods in tension or on flanges of I-beams in bending or both. All of them were also

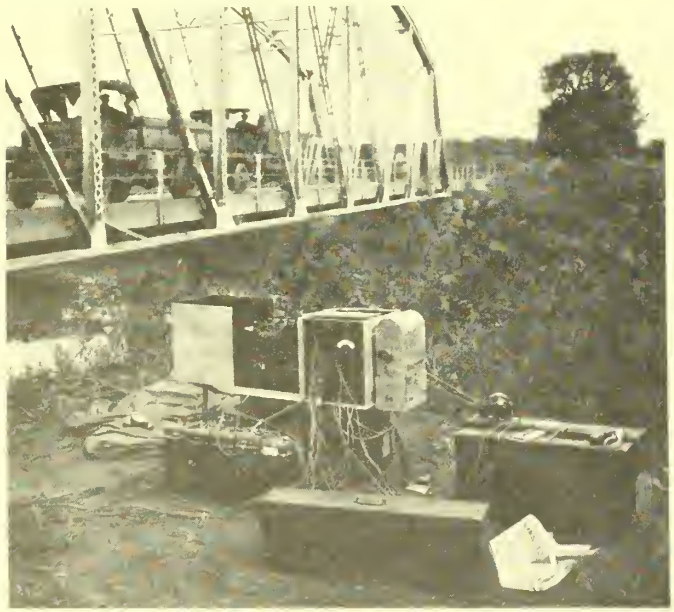


FIG. 2.—Bureau of Standards instrument and Skunk River bridge

calibrated for impact on a special impact machine, shown with a number of instruments attached in Figure 3. It consists essentially of a "hammer" weighing approximately 2,060 pounds supported so that it may be raised through known distances and allowed to drop freely upon the head of a 1-inch rod in tension. The range of motion and the deformation in various portions of the mechanism were read by means of a number of Ames dials.

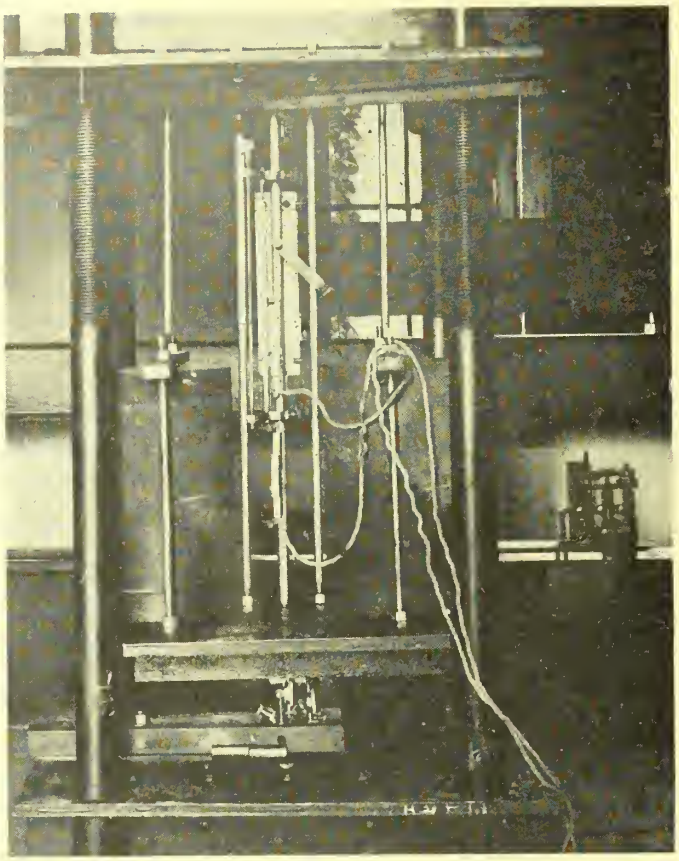


FIG. 3.—Calibration of instruments under impact

The Turneaure instrument, used in 1922 and described in Bulletin 63 of the Iowa Engineering Experiment Station, was observed to lag for small impacts and to overthrow for large ones, but its readings for intermediate impacts compared reasonably closely with those of the other instruments.

As suggested in the report of the previous year's work, the dials of the West instrument [1] were choked or damped with a view to increasing the accuracy of the instrument for high impacts. The behavior in the calibrating machine and in comparison with the other instruments on the bridges gave assurance that this could be done, but considerable care, including frequent calibration, was needed to secure consistent and correct results. The few days' use of the Bureau of Standards instrument indicated that it will surpass all of the other instruments in the number of readings that can be made with it in a given time, in ease of identification and readiness of interpretation. In general, the readings of the various instruments checked reasonably well and there is considerable confidence in the season's results.

As in 1922 the impact was delivered by solid-tired trucks with wheel loads as given on page 10. The trucks were operated at full speed (from 10 to 14 miles per hour). Observations were made of the impact set up by truck operation over the clean bridge floor and over 1 by 2 inch and 2 by 4 inch obstructions.

Table 4 gives a summary of the impact observations on various members of the several bridges. The results are given in unit stresses and in impact percentages. The values have been chosen not by

[1] For description See Bul. 63, Iowa Engineering Experiment Station.

mathematical average but by inspection, in which greater weight has been given to the runs made under the more satisfactory and more nearly typical conditions.

TABLE 4.—*Summary of impact percentages and impact unit stresses, Skunk River, Squaw Creek, and Roland bridges*

[Truck speed 10–14 miles per hour]

Member	14-ton truck					
	No obstruction		1 by 2 inch obstruction		2 by 4 inch obstruction	
	Stress	Per cent	Stress	Per cent	Stress	Per cent
	Pounds		*Pounds*		*Pounds*	
Stringers, approach span, Skunk River bridge	4,400	22	5,400	43	9,300	140
	4,100	16	4,700	32	8,000	120
Stringers, main span, Skunk River bridge	11,000	35	12,200	57	15,000	95
	10,000	20	11,200	40	13,400	70
Top chords and end post, Roland bridge	4,500	40	7,000	100	13,000	350
	4,000	25	6,000	70	9,800	180
Lower chords, Roland bridge	2,900	43	3,600	70	4,300	110
	2,600	30	2,900	43	3,900	78
First diagonals, Skunk River bridge	2,500	60	4,600	150	9,000	350
	2,300	40	4,000	120	7,500	250
Hip verticals, Skunk River bridge	[2] 4,100	[2] 17	[2] 7,500	87	[2] 11,500	175
	[3] 2,000	22	[3] 4,000	137	[3] 6,700	275

Member	Two 14-ton trucks					
	No obstruction		1 by 2 inch obstruction		2 by 4 inch obstruction	
	Stress	Per cent	Stress	Per cent	Stress	Per cent
	Pounds		*Pounds*		*Pounds*	
Stringers, approach span, Skunk River bridge	5,800	17	7,400	40	10,500	100
	5,600	12	6,700	32	8,800	75
Stringers, main span, Skunk River bridge	10,600	17	14,000	43	16,000	85
	10,000	10	12,000	34	14,500	67
Stringers, Squaw Creek bridge	6,700	21	10,400	95	15,000	180
	6,100	14	9,800	82	13,000	120
Floor beam, Skunk River bridge	8,300	10	10,500	50	11,500	68
	7,800	10	9,500	45	10,200	55
Floor beam, Squaw Creek bridge	5,000	15	6,500	50	9,000	92
	4,500	10	6,000	40	7,500	77
Floor beam, Roland bridge	6,700	17	8,000	45	14,000	150
	6,400	10	7,500	30	12,000	100
Top chords and end post, Roland bridge	5,900	29	7,400	50	10,000	115
	5,300	20	6,700	35	8,800	78
End post, Skunk River bridge	1,200	45	------	------	2,000	80
	1,000	30	------	------	1,500	30
End post, Skunk River bridge [1]	1,500	45	------	------	2,200	45
	1,200	30	------	------	2,000	30
Lower chords, Roland bridge	3,400	28	3,900	50	6,000	130
	2,900	20	3,400	33	4,500	67
North girder, west span, Squaw Creek bridge	3,000	25	3,200	30	4,500	65
	2,800	20	3,000	25	4,000	50
Do.[1]	3,000	30	3,300	35	5,000	80
	2,800	25	3,000	30	4,500	60
First diagonals, Skunk River bridge	3,500	70	------	------	7,500	200
	3,000	50	------	------	7,000	150
Do.[1]	3,350	38	6,600	140	7,000	140
	3,000	30	4,900	75	6,000	100
First diagonals, Roland bridge	7,300	48	9,700	73	12,000	140
	6,800	30	7,000	40	10,000	100
Remaining diagonals, Skunk River bridge	4,200	25	4,600	40	6,800	100
	3,800	20	4,100	30	5,800	72
Remaining diagonals, Roland bridge	2,500	58	3,400	100	4,600	185
	2,100	35	2,800	60	3,500	110
	−4,000	−75	−4,200	−91	−5,000	−125
	−3,200	−33	−3,600	−60	−4,300	−75
Hip verticals, Skunk River bridge	[2] 6,200	30	[2] 11,000	105	[2] 13,000	150
	[3] 2,400	23	[3] 3,800	85	[3] 4,800	112
Hip verticals, Roland bridge	[2] 4,100	20	[2] 5,200	40	[3] 8,500	130
	[3] 1,700	30	[3] 2,600	100	[3] 5,000	300
Post U2′–L2′, Skunk River bridge	[2] 2,200	50	[2] 3,600	45	[2] 10,500	200
	[3] 1,500	25	[3] 1,500	30	[3] 2,200	100
	[2]−2,000	−25	[2] −2,100	−45	[2] −4,000	−150
	[3]−1,800	−15	[3] −1,900	−30	−3,500	−100

[1] Two trucks in tandem, all others abreast.
[2] Average stresses in inside flanges.
[3] Average stresses in outside flanges.

Minus signs indicate reverse stresses in members.

[Upper figures are the averages of the two high stresses in each group of runs. Lower figures are the general averages]

FUTURE WORK

Ultimately the work on this problem should lead to the acquisition of data which will give in terms of impact the effect of various tires, conditions of tires, chains, etc. The immediate problem, obviously, is not to continue until all of these possibilities have been investigated for all types of floors, but so to standardize the conditions already used as to serve as a means of interpreting other conditions.

This standardization may consist in establishing a relation between the impact stress and the simultaneous force of the blow of the truck wheels on the bridge floor. After this has been done the work of the United States Bureau of Public Roads, in which by the compression of copper cylinders, use of accelerometers, etc., the force of impact blows has been determined, may be used to estimate the impact stresses in a bridge to which such impacts are delivered.

The report proposes a definite program for the continuance of the tests over a two-year period, and suggests, among other things, the construction of a special test span apart from a traveled highway on which the effect of various loads, such as a crowd of people, of livestock, a number of heavy trucks, etc., may be tested without inconvenience to the public.

THE ORGANIZATION OF THE ADVISORY BOARD ON HIGHWAY RESEARCH

The Advisory Board on Highway Research of the National Research Council announces that 40 State highway commissions have already responded to the invitation to name representatives to serve on the board. Through the State representation the highway commissions and the Advisory Board will have a medium whereby research problems may be thoroughly studied. Through this medium problems will be brought to the attention of the State departments, and the solutions will be made known.

The board, which is now quartered in the new building of the National Research Council at B and Twenty-first Streets, Washington, D. C., is a branch of the Division of Engineering and Industrial Research. It is headed by Charles M. Upham, as director.

The standing committees through which the work of research is conducted are as follows:

Committee on economic theory of highway improvement. T. R. Agg, chairman.
Committee on structural design of roads. A. T. Goldbeck, chairman.
Committee on character and use of road materials. H. S. Mattimore, chairman.
Committee on highway bridges.
Committee on highway finance. H. R. Trumbower, chairman.
Committee on maintenance. W. H. Root, chairman.
Special investigation on reinforced concrete roadway. C. A. Hogentogler, chairman.
Committee on special assignments.

The following representatives have been appointed by the several State highway commissions:

Alabama, J. H. Scruggs.
Arizona, B. J. McNelly.
Arkansas, F. A. Gerig.
Connecticut, Geo. E. Hamlin.
Delaware, W. W. Mack.
Georgia, Searcy B. Slack.
Idaho, H. W. Gregory.
Indiana, C. Gray.
Iowa, R. W. Crum.
Kansas, C. H. Scholer.
Maine, L. D. Barrows.
Maryland, Harry D. Williar, jr.
Michigan, G. C. Dillman.
Mississippi, Geo. B. Denham.
Missouri, F. V. Reagel.
Nebraska, C. M. Duff.
Nevada, H. M. Loy.
New Hampshire, Wallace F. Purrington.
New Jersey, G. Roland Moore.
New Mexico, I. E. Burke.
New York, Wm. L. Blaum.
North Carolina, C. N. Conner.
North Dakota, W. G. Black.
Ohio, A. S. Rea.
Oklahoma, Frank Herrmann.
Pennsylvania, Wm. H. Connell.
Rhode Island, G. H. Henderson.
South Carolina, Charles H. Moorefield.
Tennessee, O. H. Hansard.
Utah, E. S. Borgquist.
Vermont, Thurman W. Dix.
Virginia, Wm. R. Glidden.
Washington, E. R. Hoffman.
West Virginia, R. B. Dayton.
Wisconsin, ———.
Wyoming, W. B. Norris.

THE EFFECT OF HAUL ON THE COST OF EARTHWORK

A Study by the Bureau of Public Roads of This Element of the Cost of Excavation

By J. L. HARRISON, Highway Engineer, U. S. Bureau of Public Roads

THE most reliable statistics available indicate that the annual expenditure for highway construction is now approximately $600,000,000. Of this amount from $150,000,000 to $200,000,000 is expended on Federal-aid roads, concerning which accurate and complete data may be had. An analysis of these data discloses the fact that approximately 25 per cent of this money is spent for grading. There is no definite information as to the proportionate cost of this element on other roads but it is probably as great as on the Federal-aid roads. If this assumption is accepted, the general magnitude of the field on which this study bears—that is, the magnitude of its direct financial aspects—may be properly presented by the general statement that from $100,000,000 to $150,000,000 is being expended annually in the movement of subgrade materials as an incident to highway construction. Any information which will improve efficiency in this field has, therefore, unusual possibilities of producing tangible savings of importance, an item which should receive careful consideration prior to beginning any investigational work.

The Federal-aid road data indicate that the unit prices at which contractors accept excavation vary between rather wide limits. Omitting from the present discussion those States in which the problem is complicated by special considerations, such as removing old pavements, extra long free haul, unusually rocky land, etc., there still remain important differences in the average unit prices bid on excavation, as the following table shows:

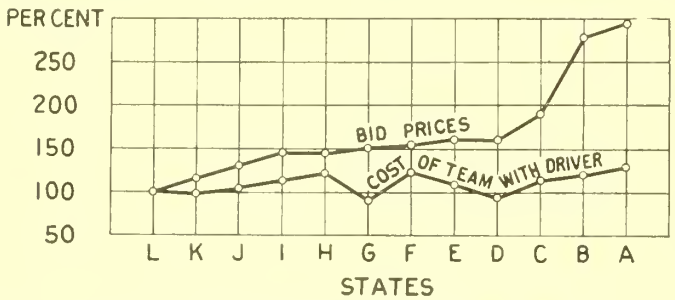


FIG. 1.—Variation in bid prices for earthwork in 12 Mississippi Valley States, compared with cost of team with driver

Average unit bid prices for excavation in Mississippi Valley States, 1923

State	Quantity under contract	Average cost of excavation per cubic yard	Per cent of lowest State average cost	Average hourly cost of team with driver	Free haul limit
	Cubic yards	*Cents*	*Per cent*	*Cents*	*Feet*
A	555,589	59	295	75	1,000
B	878,516	56	280	69	None.
C	1,253,813	38	190	65	700
D	241,604	32	160	53	500
E	2,408,704	32	160	62	1,000
F	3,196,918	31	155	70	500
G	1,849,790	30	150	51	1,000
H	2,939,685	29	145	69	500
I	3,922,329	29	145	65	500
J	1,019,805	26	130	59	500
K	3,435,439	23	115	56	400
L	4,467,174	20	100	57	200

It has been common practice to assume that such differences in cost of excavation as are revealed by this table are due in a large measure to variations in the cost of labor and teams. These are, of course, important factors in the cost at which grading work can be accomplished but it may be reasonably assumed that if these were the only important factors grading prices would vary in substantial harmony with them. Figure 1 shows the variations in bid prices as given in the fourth column of the table and also the variations in average hourly cost of a team with driver as given in the same table. This graph plainly indicates that other factors than the cost of teams and drivers are paramount in determining variations in unit bid prices.

Among other factors the equipment used and the soil conditions encountered suggest themselves as possibly having enough influence on cost to offer an explanation of the differences not accounted for by variations in the cost of team time. As bearing on the problem in hand, these can, however, be set aside, as the States shown in the above table have been selected from the Mississippi Valley where fresnoes, wheel-scrapers, and elevator graders are the common earth-moving equipment and where soil conditions, while by no means uniform, are, on the whole, favorable to their use.

In this study it has not been considered necessary to differentiate sharply between the several kinds of material, such as sand, loam, and clay, as they usually occur in the Mississippi Valley, and which are generally classed as common excavation. It is true that in a given time and for a given distance a normal fresno or wheeler outfit would probably move a somewhat different quantity of one material than of another, but these differences do not appear to be sufficient to have any controlling effect on the questions here involved. Nor should the location of the material—that is, whether it is in one State or another—have any bearing on the quantity which can be moved a given distance in a given time by such an outfit. Neither small variations in the character of the material nor its location with reference to State lines should, therefore, enter as important factors in establishing average bid prices, especially when the quantities considered are large and contain all these materials as they do when they result from averaging a considerable number of jobs in each State.

The depreciation, wear and tear, and breakage of equipment are large elements in the cost of moving earth. So also are loss and depreciation of stock. They are of small importance here because there is no reason for supposing that mules or equipment depreciate faster in Dakota than in Texas, and even if it could be shown that some difference in rate of depreciation does exist it is not at all likely that this difference would be great enough to have any large influence on bid prices.

LENGTH OF HAUL A CONTROLLING FACTOR IN THE COST OF EARTHWORK

Besides these factors, there is in all excavation work the factor of length of haul. This study has brought

out clearly the strong influence of length of haul on the cost of excavation. Other factors such as the character of the soil, grades, and prevailing weather conditions, over which the designer of the road has practically no control, may operate to increase or decrease the cost of the earthwork; but the length of haul appears in general to have a more marked influence than any of these and it is an element which can be definitely controlled in advance by the designing engineer. That an increase in the length of haul which doubles the time required per trip in moving earth from the excavation to the fill will double the cost of moving it may be accepted as a foregone conclusion. While the study confirms the accepted practice in regard to the kind of equipment best adapted to hauls of various lengths, it appears to be well established that adaptation of equipment will not enable designing engineers or contractors to disregard distance as a primary consideration in the cost of moving subgrade materials. By showing that positive and material savings can be effected by reducing the average haul it indicates that engineers may reconsider with profit certain of the practices which are accepted as fundamental in grade location, especially the practice of balancing cuts and fills by long end or longitudinal hauls. Consideration must, of course, be given to the use of excavation in embankments where the cost of overhaul is less than the cost of wasting plus the cost of borrow.

TEAM TIME AS A BASIS FOR STUDYING COSTS

Whenever an effort is made to determine the money cost of moving a yard of earth an astonishing number of variables appear. In the first place, the amount of earth that can be moved in any given time varies with the distance it is moved. The amount which can be moved per unit of time also varies somewhat with such conditions as grade, weather, temperature, etc., but whatever affects the output per unit of time affects the unit cost.

Moreover the money cost of team time varies not only from job to job but from week to week on the same job. Hay and grain, the principal items in the cost of maintaining stock, fluctuate in price over a wide range, and other elements which enter into the cost of delivering an hour of team time on any job are equally inconstant. Therefore, to approach the problem of determining why unit bid prices are higher in some States than in others by an analysis of unit costs in dollars seemed unwise as the number of fluctuating elements left no apparent basis for even a day-to-day comparison of results and made the possibility of any valuable comparison as between different jobs very difficult of attainment.

For these reasons, instead of using the cost in money as a basis for comparative studies, an effort was made to develop the time-distance basis. The theory that underlies this basis of comparison is that an average mule team pulling any given variety of earth-moving equipment ought to travel as fast in the Dakotas, Kansas, or Iowa as it will travel in Texas or Louisiana if working in the same sort of material. It was recognized that high temperature in the South or low temperature in the North would probably affect the rate of travel, but it was, of course, apparent that these influences could be avoided by using a little care in the selection of the season for making field studies.

In this connection it might be observed that when a time-distance basis is adopted in the study of earthwork the problem is reduced to one of obtaining information as to how long it normally takes to move a unit of excavation a given distance, or, to reverse the statement, how many units of excavation can be moved over any distance in a unit of time. The relationship as between different jobs and between regions is then expressed in units of team time, standard equipment and a uniform hook up being assumed. In the application of the data secured to estimating either by engineers or by contractors, the value of team time at any point can be determined readily enough by an examination of local rates of pay, feed costs, etc., having at all times in mind the fact that team time, as here used, includes the driver and all animals used in operating a unit of equipment.

The selection of this basis for comparison makes it a simple matter to study such elements as material, grade, temperature, white versus colored drivers, etc., for, given a standard hook up and ample time, it is possible to study these or any other factors by segregating the data which diverges from the normal only as to one factor and accumulating it until significant averages can be obtained. The results which have been secured by the use of this basis for study justify the impression that it will offer a ready solution for many grading problems which have in the past seemed more or less intangible.

THE FIELD WORK

The first step in making these studies has been to examine going construction projects with a view toward selecting for extensive study those where the amount of cut is large enough to enable the engineers to obtain appropriate data and where the equipment is standard. Jobs where the work is light have so far been avoided because, under such conditions, the outfits are apt to be so spread out that the collection of data is difficult. Jobs where the equipment is not standard or which include two or more types of equipment in the same group have also been avoided, as, under such circumstances, it is difficult to determine quantities with accuracy. In short, the first work has been to get data concerning main features, leaving special conditions for later studies. When the project is determined upon the established practice is to select and carefully cross-section a cut. When work in this cut is begun a record is kept of the number of hours that are required in moving the material and the distance it is moved, together with other pertinent data. As often as desired while the work is in progress, new cross sections are taken and quantities calculated. Time studies are made daily to show the average time required to make a round trip including loading and unloading, the average distance the earth is moved being determined with such care as is possible where a working outfit must be studied without being disturbed.

INFORMATION DEVELOPED BY THE STUDY

Figures 2 and 3 show the relation between length of haul and time required per trip by No. 2 wheel scrapers and 4-foot fresno scrapers, respectively. They represent the average relation as found in Kansas, Missouri, Oklahoma, and Texas, in which the performance is similar.

The graphs appear as straight lines, indicating, for the studies made, that the average rate at which teams travel does not change materially with the distance the load is moved. As extended, they cross the horizontal axis some distance to the right of the zero point. This distance represents the average time required for loading, unloading, and turning, together with the minor delays incident to operations of this nature. If the distance is short, material can sometimes be moved by fresno from the ditches to the embankment within the time here set down as the average for loading, unloading, and turning. As this fact has only a limited bearing on the matters discussed in the article, it is merely noted here for subsequent amplification and discussion.

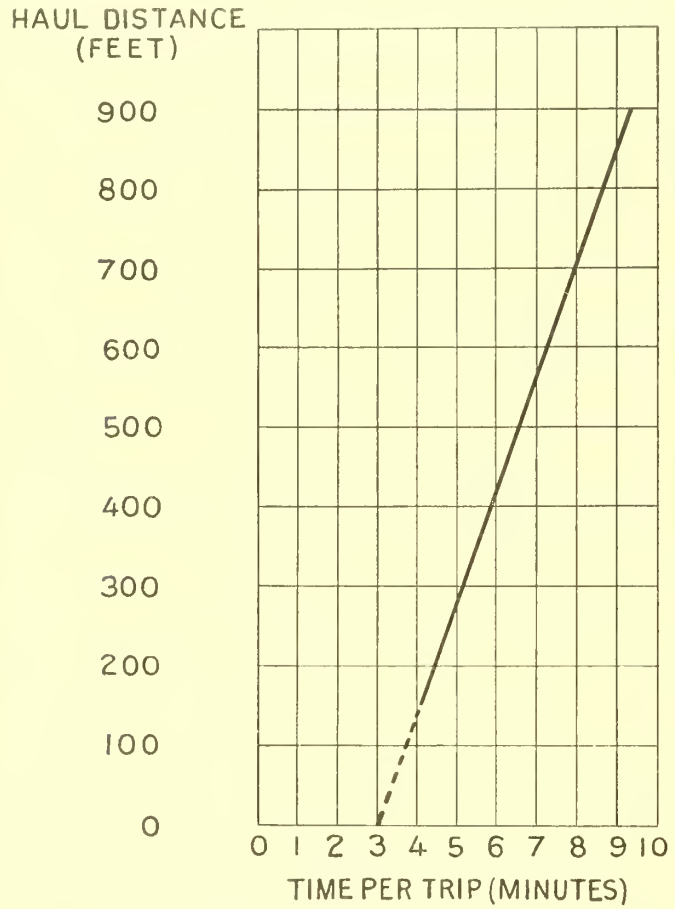


FIG. 2.—Relation between length of haul by No. 2 wheel scraper and time per trip

Translating the general time-distance graphs into mathematical formulæ we have, for fresnoes,

$$t=1.2+0.0101D$$

in which,

t=time per load in minutes and
D=distance load is moved in feet

The factor 0.0101 is governed by the rate at which teams have been found to move under average conditions. The constant 1.2 is the time consumed in loading, unloading, and turning.

For wheelers the formula similarly developed is

$$t=3+0.0071D$$

Repeated measurements made in the field indicate that the average load for normally operated fresnoes ranges from slightly over one-third cubic yard on short hauls to slightly under one-third cubic yard on long hauls. The load of the ordinary wheel scraper is found in practice to vary in about the same manner with an average of 0.4 cubic yard per trip. The studies on the standard 1½ cubic yard wagon operated in connection with an elevator grader are not as complete as for wheelers and fresnoes, but they indicate that wagons operated with elevator graders average about 1¼ cubic yards per trip. Reducing the above formulæ to a 1-cubic yard basis in the light of these data, we have, then, these general formulæ:

For wheel scrapers, $T=2\frac{1}{2}\ (3+0.0071D)$
For fresnoes, $T=3\ (1.2+0.0101D)$
in which, T = time required to handle 1 cubic yard of material

In a further analysis of the time-distance curves it may be noted that the graphs for the fresno and wheel scraper are, in a general way, directly comparable. A fresno outfit, as operated by the usual small contractor, includes the foreman (usually the subcontractor himself), the plow team with driver and plow holder, and six to eight three-mule fresno teams with drivers. The drivers customarily load and dump as well as drive. A wheel scraper outfit of this size has a foreman (generally the subcontractor), a plow team with driver and holder, a two to three-mule snatch team and driver, a loader, a man on the dump and from 6 to 10 two-mule wheelers. The items of supervision and plowing are so nearly the same, whether wheelers or fresnoes are used, that consideration of them may be omitted. As a rough generality the commercial value of teamster's time is about the same as that of a two-mule team The fact that a wheeler outfit uses a snatch team with

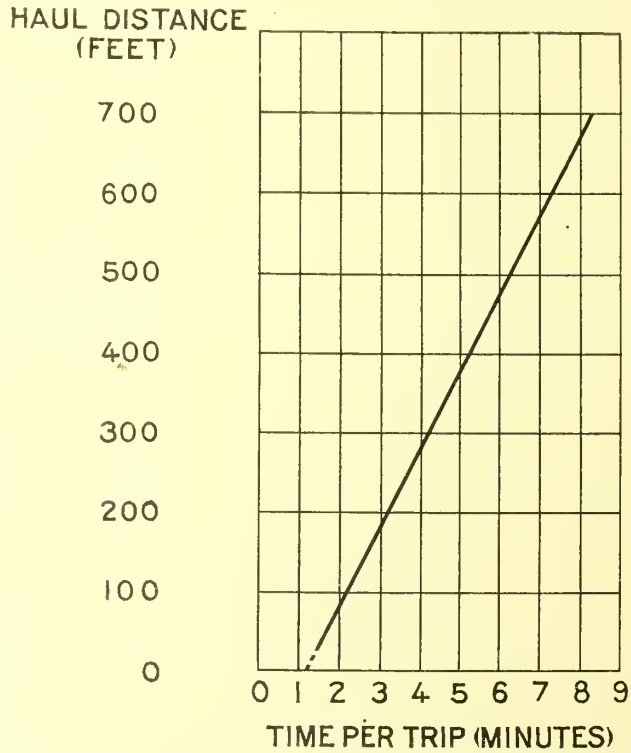


FIG. 3.—Relation between length of haul by 4-foot fresno scraper and time per trip

driver, a loader, and a man at the dump (none of these being customarily used with a fresno outfit), which, in a general way, is equivalent to eight mules, makes up for the extra mule on the fresno. From this standpoint the cost per unit of time of operating a fresno is so nearly the same as the cost of operating a wheel scraper that equality may be assumed. It therefore results that the point of equal operating cost may be obtained by equating the two formulæ:

$$3\ (1.2+0.0101D)=2\tfrac{1}{2}\ (3+0.10071D)$$
$$\text{or } D=310 \text{ feet}$$

The importance of this deduction lies in the fact that it shows that wheelers can not be successfully operated in competition with fresnoes except at hauls exceeding 300 to 350 feet, and therefore that lower bid prices can not be secured by resorting to the use of this type of equipment. While both on the basis of mathematical development and observed field practice it appears that fresnoes should be laid aside and wheelers used at this haul distance, contractors claim that fresno hauls as long as this are hard on the stock and so must be used sparingly. The field observations confirm this claim. Moreover, from the graph for fresno work it is a simple matter to show that the teams are under full load almost twice as many minutes per hour on a 300-foot haul as they are on a 100-foot haul. It therefore appears that the statement that a 300-foot fresno haul ought not to be attempted over any extended period is sound.

The data which have been secured in regard to wagons operated in connection with elevator graders have reasonably established the fact that wagons move at about the same speed as wheel scrapers. From the limited number of studies on this type of equipment it has not yet been possible, however, to derive a mathematical statement which can be equated with the formula given for the fresno and the wheeler in order thereby to develop the haul distance below which this style should be laid aside. However, enough data have been collected to warrant the statement that while an elevator grader outfit of the usual pattern handles the longer hauls as cheaply as the wheel scraper, it has not been possible to show that it is effective in so reducing costs that it is comparable with the fresno on hauls much if any below 300 feet.

RESULTS OF THE STUDY

The net result of these studies, so far as they bear on the specific problem under discussion, has been, then, to show that there appears to be no means of handling earth which eliminates the factor of distance moved as a dominant element in cost. At certain hauls a wheeler will move earth cheaper than a fresno and for the longer hauls—over 300 feet—it appears probable that the elevator grader moves earth somewhat more cheaply than either of the other forms of equipment. But no form of equipment has been found in common use which enables contractors to disregard distance as a primary consideration in the cost of moving subgrade materials. The time-distance graphs make this fact very clear. In terms of fresno operation, where the movement can be kept within 75 feet, the time required may be kept within two minutes per load, while, if the movement be 300 feet, it will require slightly over four minutes. But a grading operation that takes four minutes of team time costs the contractor twice as much as an operation requiring two minutes of team time.

It is not necessary to expand this statement to great length to justify the conclusion that grading costs may be reduced by a general modification in design which will reduce the haul distance. The State designated as L in the table consistently shows unit bid prices for excavation which are relatively low. In the light of the conclusion above reached, it is interesting to observe that two elements in the L subgrade design contribute to this, viz, the cross section and the low free-haul limit of 200 feet. This cross section, shown in Figure 4, somewhat generalized in order to make its variable elements clear, differs from that in general use in other parts of the country in that it permits considerable side balancing of quantities instead of depending on a balance by end or longitudinal haul. This is accomplished by widening, sometimes accompanied by deepening the ditches so that a large percentage of the material needed for fills can be secured from alongside. In coupling a practice of this sort with a low free-haul limit engineers find it desirable to reduce cuts to a minimum. The research work done at the Iowa State College seems to have established the fact that rolling grades are of little disadvantage to the users of highways, a proper maximum being observed. It appears, then, to be a natural conclusion from the foregoing that at least on projects with average excavation quantities lower bid prices may be secured through such adjustments of design as may be necessary in order to secure short hauls.

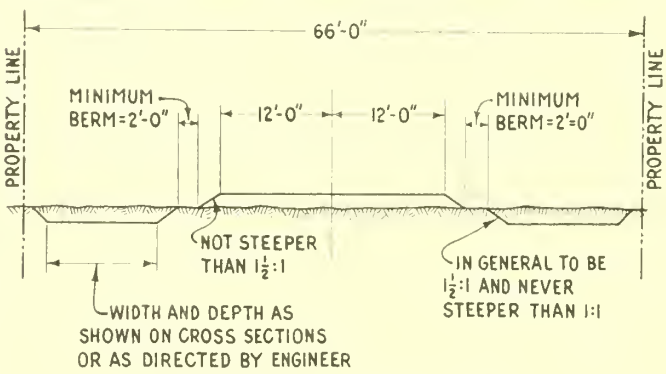


FIG. 4.—Generalized cross section used by State L

The time-distance graphs indicate that the current practice of balancing cuts and fills by long end or longitudinal haul is expensive. Many States have so-called free-haul limits of 500 feet or more with design practices established in accordance therewith. In those cases where the free haul is as low as 500 feet but is calculated from center of mass to center of mass, the actual maximum haul will not infrequently run over 1,000 feet. Of course, the amount of material moved 1,000 feet is not large but it requires almost exactly twice as long to deliver it this distance by wheeler as it does to deliver it a distance of 300 feet. At a delivery distance of 300 feet the cost of wheeler work and fresno work is practically the same. A fresno longitudinal haul of 300 feet, in turn, takes about twice the time generally required for side borrow work. The effect of design based upon long free-haul limits on bid prices when compared in this way with short-haul work, is apparent.

RHYTHMIC CORRUGATIONS IN GRAVEL ROADS

A Study of the Nature of "Chatter Bumps" and Their Relation to Traffic

By GEORGE E. LADD, Economic Geologist, United States Bureau of Public Roads.

WHAT is the nature of the rhythmic corrugations which occur in gravel roads in all parts of the country and which are a source of inconvenience if not an actual menace to traffic? How do they originate? Do they migrate or are they stationary? What is their relation to traffic? What relation have they to the kind of gravel used or the method of construction? What are the best maintenance methods with which to combat them? These and other similar questions have been investigated by the United States Bureau of Public Roads in the States of Maine,[1] New Hampshire, Vermont, Massachusetts, Connecticut, New Jersey, Michigan, and Wisconsin. Detailed studies of gravel-road construction jobs were made in four States with the object of keeping them under observation in the future. Gravels were sampled and tested, methods and costs of construction and maintenance were studied, and notes were made bearing on the traffic limitations of gravel roads.

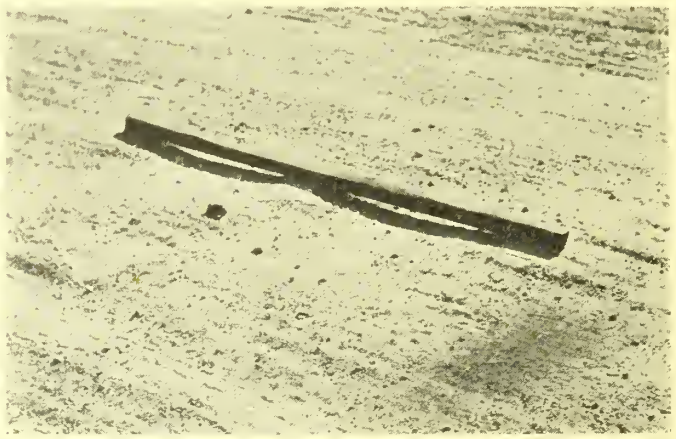
Corrugations resulting from impact on a road which is frequently sprinkled

NATURE AND ORIGIN OF THE CORRUGATIONS

Gravel-road corrugations are analogous in form to those occurring in bituminous roads, excepting, that the former develop more uniformly and extensively. If not removed by maintenance they develop until each corrugation crosses the whole course of traffic and each series of them develops along the road until the next series is overtaken. The distance from crest to crest averages about 31 inches, and crests are rarely found either less than 25 inches or more than 35 inches apart. Any considerable variation from the average appears to be confined to the area where one series as it develops along the road overtakes another. The maximum height from bottom of trough to top of crest is 1½ inches. Any greater height than this was found in the field study to be associated with the beginning of pitting and raveling. These figures appear to be true of corrugations regardless of the nature of material in which they occur or the methods by which they develop. People traveling by automobile over roads so corrugated, not taking into consideration the speed at which they are passing over them, almost invariably estimate the distance from center to center of crest at 8 to 12 inches. A large number of highway engineers who have never given the problem special attention fall into the same error. The facts, however, as stated, were determined by observation on hundreds of miles of corrugated gravel roads in various States.

Inquiry among road builders as to the cause of gravel road corrugations resulted in the discovery of an amazing variety of explanations, some of which were very ingenious. The majority of replies erroneously attributed these corrugations to maintenance methods, particularly the use of the drag. The most ingenious explanation was offered by a man working on a corrugated road on the coast of New Jersey. He affirmed with positiveness that they are caused by the action of waves on an adjacent beach, transmitted to the road by earth vibration. As a matter of fact, these rhythmic corrugations develop in two ways, or through a combination of the two. They originate in vertical oscillations of rapidly moving automobiles, produced by obstacles, rough places in the road, or sharp depressions, and to this extent might in individual cases have their origin in an exaggerated case of poor dragging. They are, however, too widespread and too independent of dragging to justify in any general way their being attributed to this factor as a cause. Methods of their development which, it is repeated, may in some cases be combined, are as follows:

First method.—They are formed by the kick-back of surface materials arising from the spin of one or both of the rear wheels of automobiles as they descend after a bounce over some obstacle or depression. In the State of Maine, where gravel roads are maintained by keeping a loose surface brushed over the road by means of scrapers, drags, and planers, corrugations were observed from their incipiency over a very extensive area. It was noted frequently that where cars bounded over obstacles or rough places in the road, a set of corrugations was started by the action of one



Typical corrugation, showing spacing and depth

[1] The investigation was made during the summer of 1921 by the writer, leading a party consisting of F. D. Hurwitz, E. F. Strickler, and two temporary assistants.

rear wheel only. As the wheel descends with accelerated rotation it momentarily kicks back a portion of the loose material of the road surface. Spring action causes a second and third like phenomenon. As many as five of these mounds have been seen in sequence. By repetitions of this action elliptical mounds of loose material are formed the longer axes of which are at right-angles to the axis of the road. As traffic continues the action is transferred to the other rear wheel and a parallel set of these mounds is formed. The weaving in and out of cars rapidly connects the pairs of little mounds into a ridge and these ridges in time become continuous across the entire road. The shifting of cars about the center of the road evidently carries corrugations across it, so that they are picked up by traffic going in the opposite direction and they ultimately form a continuous series of crests and troughs extending from border to border. The writer has never seen two independent series on opposite sides of the road, where ridges do not coincide. It would appear, therefore, that they develop across the road faster than a series can multiply and advance along the road.

An interesting fact about their occurence is that they are never found on steep hillsides. They may occur on the lower slopes or on broad tops, but they are not found either on the ascending or descending steep slopes. This tends to verify the explanation of their origin, which has been given. Going up the steep hill wheels of cars cling more closely to the road and on the descending side the cars coast. In neither case is there opportunity for the driving wheels to throw back surface materials in the manner indicated.

Corrugations of this type are never well bonded and compact. After weeks of heavy traffic they may be scraped away by the hand.

Second method.—Whenever the clay bond in a gravel road is sufficient in quantity and has sufficient moisture to give the aggregate a certain plasticity, and especially where the gravel particles are well rounded, series of corrugations develop which correspond in measurements to those described above. Their origin, however, is different from that of corrugations of the class decribed above, in that they result from the squeezing of a plastic mass, and are produced in a way similar to the development of waves in improperly mixed bituminous roads. Such corrugations have been observed in northern States where glacial gravels are used, but there they are rather a result of a much too high original bond or the scraping of ditch dirt on the road. This type of corrugations may be best observed in New Jersey, especially on the Shore Road in and near Long Branch. Here the roads are built of highly water-worn gravel of small sized particles so rounded and polished that a clay binder, as low even as 10 per cent by weight, makes possible a plastic condition of the whole, especially when sufficient moisture is added. The mositure on certain roads in question is furnished by daily sprinkling, in others by rainfall or by capillarity. Similar phenomena have been noted in roads built of cinders.

Rhythmic corrugations of this type result from impact, and, unlike the case of the first type discussed, the front and rear wheels of automobiles are equally active in their development.

DO THEY MIGRATE OR CHANGE POSITION?

Before the investigation was begun it was assumed that corrugations would slowly advance in the direction of traffic. Those, however, originating by the first method discussed appear to remain fixed where they originate. They are kept down in height by the action of wheels passing over them, and more especially by the limit of bound of cars, but they seem to move neither forward nor backward. At a number of places where they developed early in July reference stakes were driven on one side of the road, and they were kept under observation until late in the fall. No evidence of migration was noted. If corrugations move at all it would seem that they would at least swing on an axis located near the center of the road. This is exactly what happens with the impact type which originate in the more or less plastic varieties of gravel road. Many cases of this swinging on an axis were observed and photographed. This phenomenon is conspicuous where corrugations 6 or 8 feet in length cross the center of the narrow roads of a semiplastic nature. The writer has seen these swinging movements progress from a position at right angles to the road to one departing but a few degrees from the axis of the road.

In New Jersey an arrangement of corrugations was found which for a time was very puzzling. They were "herringbone" corrugations. They approached the center of the road from both sides at an angle of about 60 degrees with its axis, making a series of V-shaped markings centering along the middle of the road. That these corrugations were originally at right-angles to the axis of the road is evidenced by the fact that where the road is crossed by a concrete walk, a fixed obstacle, traffic had developed normal corrugations in its immediate vicinity on the take-off side but not on the approach side. At a distance of 12 or 15 feet away, however, they became angular and assumed the positions described. In seeking a solution of this phenomenon the writer found conditions as follows: The road is 60 feet in width with a very low crown. It is built of gravel consisting of very highly rounded and polished pebbles and sand and about 10 per cent of clay-silt bond. The road is sprinkled with water throughout the summer season to prevent dust. During the summer it is subject to traffic which rises on holidays to more than 10,000 cars a day. It is close to the shore and overlooks the surf, but automobile passengers are prevented from seeing the beach itself because of a broad boardwalk on the shore side. Considering the fact that here is a great bathing resort, and that the average automobilist enjoys the sight of surf bathing, it was assumed that when traffic was not so excessive as to confine cars to double rows in both directions, that there would be a drift of travel toward the shore side, traffic going north as near to the boardwalk as possible, and on the other side, going south, as near to the center of the road as possible, in both cases to obtain the best possible view. Such a drift of traffic on this road would account for the extraordinary swing of the corrugations observed. Whether or not this theory gives the correct explanation of the behavior of corrugations on this road, it at least accords with the facts and is a possible explanation.

THEIR RELATION TO TRAFFIC

Gravel roads subject to a traffic of not more than 200 or 300 cars per day remain practically free from corrugations if occassionally dragged. As soon, however, as traffic reaches 400 to 450 cars per day, corrugations develop very rapidly. In the State of Maine, where tourist traffic is large during the summer months, and its rate of increase is well known, the State highway officials can predict almost to a day when corrugations will begin to develop on certain roads. The traffic limitations of gravel roads in general are determined by the intensity of maintenance required and by its cost. Sufficient data have not yet been gathered on this subject. A highway engineer's handbook, referring to the subject of gravel roads, states that they are not fit for traffic exceeding 100 cars a day, but in Maine, Connecticut, New Hampshire, and other States they are successfully serving a traffic of from 700 to several thousand cars per day. In Wisconsin and Michigan gravel roads are maintained with comparatively smooth surface, so that automobile travel is comfortable and satisfactory on roads serving from 1,800 to 3,200 cars per day, with a maintenance cost ranging from $280 to $320 per year, or from $600 to $700 per year when the roads are also treated with dust preventives.

Maintenance costs on gravel roads in various States have been reported to the writer as ranging from $1,000 to $3,000 per year. No satisfactory analyses of such costs have been made and it is presumed that a considerable portion of them really belong to improvement and reconstruction.

THE RELATION OF RHYTHMIC CORRUGATIONS TO KINDS OF GRAVEL, MIX, AND METHODS OF CONSTRUCTION

The fact that these corrugations are so general and occur in all sections of the United States makes it evident that nearly all gravel roads, if they serve sufficient traffic, are subject to the development of this nuisance and menace, although the methods of construction and kinds of gravel vary widely. The situation, however, is not hopeless. Several highway officials have expressed the opinion that less corrugation trouble is found where the gravel is angular than where it is composed of highly rounded particles. It is also claimed that where the road is constructed of gravel more uniform in size than pit run, and especially where all particles over 1 inch in size are excluded, that corrugations are slower in developing and more easily eliminated by maintenance methods. In northern New Hampshire some roads are built of so-called gravel which has resulted from the decomposition of granite rocks and is full of angular quartz. The roads built of this material are said to give better service so far as corrugations are concerned than those built in the same State of glacial gravels, or those resulting from disintegration of schistose rock. It is claimed in Wisconsin that the best service given by gravel roads, so far as corrugations are concerned, is found where pit-run gravel is passed through a crusher and only the material which ranges in size from 1 inch down is used. Highway engineers, notably some in Wisconsin and Oregon, object to any clay binder in the gravel and prefer only the fines produced by crushing and such as result from surface wear.

During the writer's investigations in the State of Maine it was noted that roads with a relatively high percentage of clay-silt binder were comparatively free from corrugations. On one road on which rhythmic corrugations were well developed, a patch near the center of the road, about 70 feet long and varying from 2 to 4 feet in width, was uncrossed by corrugations and had a hard, smooth surface. Examination showed that this patch had about 5 per cent more clay-silt bond than the surrounding road.

On the gravel road between Waterville and Bangor, Me., corrugations develop rapidly during the summer traffic, and are kept down only by constant maintenance for almost its entire length. Comfortable travel on this road was made possible only by continuous use of drags and planers. One short section of the road, however, was practically free from corrugations throughout the summer. This section was built of a softer gravel which was high in clay-silt bond. Unfortunately, however, while this section of road gives satisfaction during the summer months, it is said to be nearly impassible in the spring. Thus it may be that an all-year gravel road that will not require intensive maintenance can not be built in States subject to heavy freezing and wet seasons.

This problem remains for future solution and must be worked out by a combination of field observations and laboratory tests. In this connection it may be stated that the present standard tests of gravel are unsatisfactory in that they do not include determination of the nature and qualities of natural binder originally in the gravel. The so-called cementing value test has to do only with the products of abrasion of individual gravel particles. While it is important to determine this factor, there must also be a determination, in addition to the facts brought out by mechanical analyses as to the quantity of clay-silt binder, of the cementing value of this fine material.

THE EFFECT OF DUST PREVENTIVES AND SURFACE BINDERS

A preliminary study has been made of problems of surface-treated gravel roads. Matters of methods, costs, and service have been studied to some extent in certain States. Surface binders in a general way prevent development of corrugations, and where these do occur on binder-treated roads it has been demonstrated that they result from excessive quantities of bituminous material. This statement does not refer to dust-prevention treatment, and is perhaps not true of lignin-treated roads. Some States use both tar and oil binders, some will use only oil, and others only tar. One State at least uses lignin almost exclusively. Dust preventives have been shown to fulfill the primary requirement. Their effect on corrugations, however, is not yet clear. It is not unlikely that they delay the development of rhythmic corrugations of the first type discussed above. It is clear, however, that they do not prevent their development. They successfully lay dust for long periods in spite of heavy traffic, and they lengthen the life of a road by retarding loss of surface material.

MAINTENANCE METHODS

The maintenance required by a gravel road depends, of course, largely upon the amount of traffic to which it is subjected. If this does not exceed 200 or 300 cars per day, occasional dragging, especially after rains, will keep the surface in good condition. As traffic increases, however, surface maintenance must be practically continuous, and even then dragging or scraping will not keep a road free from corrugations. The planer, a simple machine familiar to all highway engineers, is employed in addition to the drag and road scraper.

(Continued on page 22)

OILED EARTH ROADS ON LONG ISLAND

By A. T. GOLDBECK, Chief of Division of Tests, U. S. Bureau of Public Roads

A typical oiled earth road on Long Island

ONE of the very objectionable features of an untreated sand-clay or top-soil road is the dust created by traffic when the road surface is dry. Another objection is the expense of constant dragging to keep the surface in a smooth condition. Under some conditions the rate of wear of such roads amounts to as much as an inch a year and additional material must be placed on the surface to replace that worn away. On Long Island these objectionable features seem to have been overcome by the use of oil applied to the surface in a particular manner.

The writer inspected a number of these roads on the southern part of the island, extending from Bay Shore to East Hampton. The soil in general is of a sandy nature and the roads which have been treated are quite typical of sand-clay and top-soil construction. It was stated that the best results are obtained when the material is of such a nature that it is well bonded together. It must be capable of supporting heavy truck loads under ordinary weather conditions without appreciable indentation under the wheels. If the soil is practically pure sand it will be necessary first to bond the surface with a clay or loam admixture, and similarly if the soil is clay it should be treated by the addition of an admixture of sand. In Table 1 analyses of soil samples taken from untreated roads compared with typical sand-clay mixtures will give an idea of the nature of the surfaces treated.

TABLE 1.—*Analyses of samples taken from road surface before oiling, compared with typical sand-clay mixtures*

	Sample No. 1	Sample No. 2	Characteristics of typical sand-clay surfaces		
			Class A, hard	Class B, medium	Class C, soft
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Retained on 20-mesh	3.1	3.4			
Passing 20, retained on 60-mesh	38.2	53.9			
Passing 60, retained on 100-mesh	6.3	8.5	45–60	30–45	20–30
Passing 100, retained on 200-mesh	3.7	3.5			
Total sand	51.3	69.3	65–80	60–70	55–80
Silt	30.5	20.1	0–15	10–20	10–20
Clay	18.2	10.6	9–18	15–25	10–25
Total	100.0	100.0			
Total sand (retained on 60-mesh)	41.3	57.3			
Coarse material (over 10-mesh)	2.8	0.5			

It will be noted that sample No. 1 is not of as high quality as desired even for a class C sand-clay road, while sample No. 2 should make a first class sand-clay surfacing. These analyses will serve to show the characteristics of typical roads which have been oil-treated on Long Island and will be useful for comparison with sand-clay or top-soil roads in other localities where oiling is contemplated.

METHOD OF TREATMENT

The road surface is first graded to a low crown with a road machine. The oil is then applied cold with a pressure distributor at the rate of one-third to four-tenths gallon per square yard. It is then covered uniformly with sand broadcasted with shovels from piles along the side of the road. Only half of the road is treated at one time and this is immediately covered. As soon as the covering is completed traffic is allowed on the surface and treatment is then given to the other side of the road. Within about two weeks the traffic irons down the surface and gives it the appearance of sheet asphalt. Dragging is sometimes resorted to several days after oiling to aid in obtaining a smooth surface.

During the first year the common practice is to apply two treatments of oil. The following year sometimes one treatment is given and sometimes no treatment is necessary. Whenever the condition of the surface becomes uneven, blading is resorted to and the surface is brought back into shape. Every four or five years it becomes necessary to completely scarify and harrow the surface in order to bring it into first-class condition. Scarifying is done to a depth of 6 or 8 inches and the underlying soil is mixed to a considerable extent with the treated surface. The scarifying is not difficult, since even at the end of five years the oiled surface remains in a somewhat plastic condition. After scarifying and harrowing a road machine is used for shaping the surface, and traffic is allowed to consolidate it. A fresh treatment of oil and cover material is then given in the manner followed on a new road.

Broadcasting sand covering immediately following the application of oil

The oil used on the roads inspected has the following characteristics:

Specific gravity, 25° C./25° C.	0. 9587
Flash point (° C.)	58
Specific viscosity, Engler (25° C.)	109
Loss, 163° C., five hours (per cent)	24. 39
Characteristics of residue, viscous liquid, slow flow.	
Consistency of residue float test, 50° C. (seconds)	72
Total bitumen (soluble in carbon disulphide) (per cent)	99. 9
Organic matter insoluble (per cent)	. 0
Inorganic matter insoluble (per cent)	. 1
Per cent of total bitumen insoluble in 86° B. naptha	13. 5

Appearance of surface a few days after treatment. Traffic soon irons out the unevenness

NATURE OF THE ROAD SURFACE

When viewed from a moving automobile the surface looks not unlike sheet asphalt or in some cases like bitulithic when the original material contains large particles. The roadway is very readily cut with a knife and the samples thus obtained seem very much softer than sheet asphalt. A heavy touring car creates slight cuts in it and the tire markings are readily imprinted in the surface when the car moves slowly. In spite of the plastic nature of the surface, however, it does not seem to wave or shove and the shallow grooves formed by vehicles do not become deep, probably because of ironing out by traffic. The low crown on these roads is advantageous as it undoubtedly encourages traffic to use the entire width of the roadway. Surfaces six years old have retained their plastic condition and it is probable that they can be maintained indefinitely with retreatments at proper intervals. Such surfaces might have an oiled depth, owing to the building-up process of scarifying and retreatment of 3 to 6 inches, although the average depth probably was not over 1½ or 2 inches on many of the roads inspected. The depth of treatment naturally depends on the length of time the roads have been under treatment.

TABLE 2.—*Analysis of samples of Long Island oil-treated surface*

	Sample No. 3	Sample No. 4
	Per cent	*Per cent*
Bitumen extracted	8. 3	7. 2
Passing ¼-inch screen, retained on 10-mesh sieve	10. 5	9. 0
Passing 10-mesh sieve, retained on 20-mesh sieve	9. 0	9. 0
Passing 20-mesh sieve, retained on 30-mesh sieve	17. 0	19. 5
Passing 30-mesh sieve, retained on 40-mesh sieve	14. 9	15. 5
Passing 40-mesh sieve, retained on 50-mesh sieve	17. 1	13. 5
Passing 50-mesh sieve, retained on 80-mesh sieve	11. 5	9. 5
Passing 80-mesh sieve, retained on 100-mesh sieve	3. 0	2. 5
Passing 100-mesh sieve, retained on 200-mesh sieve	3. 5	3. 5
Passing 200-mesh sieve	5. 2	10. 8

NOTE.—Sample No. 3—Bayport, L. I., road 3 years old, three applications amounting to 1 gallon per square yard.
Sample No. 4—Sample taken from road surface just after scarifying and before retreating.

Some of the roads carry a very heavy touring traffic. It is stated that in some cases the number of vehicles runs up to about 5,000 and 6,000 per day during the summer season. Several heavy trucks were also noted on sections of these roads.

It seems that this treatment offers an ideal solution of the dust problem on sand-clay and top-soil roads. Macadam, gravel, and shell roads have likewise been treated successfully by this method. The cost of maintenance is only slightly more than the average cost of maintenance of the ordinary earth road in Iowa and is less than the cost of maintaining gravel roads. The riding surface presented is superior to that of either of these types and is entirely free from dust. Wherever sand-clay or top-soil roads are used, oil treatments made in the manner described undoubtedly merit a thorough trial to determine whether or not they are advantageous under local conditions. It is stated that between 1,300 and 1,500 miles of roads on Long Island have now been oiled.

(Continued from page 20)

Planers are of different types. Those used in some States have a single cutting blade, while those used in other States have several shorter blades so arranged as not only to cut down corrugations but to distribute and rearrange the surface materials. Apparently, the ideal machine for eliminating corrugations has not been devised. The planer certainly does not always entirely remove them, some of the bases of ridges being left, so that traffic may in 24 hours after the planing of the road reproduce the corrugations in original form and position. Even resurfacing with 1½ or 2 inches of gravel following planing of the road will not prevent the immediate recurrence of corrugations which have not been entirely eliminated by planing and dragging. In Wisconsin some gravel roads subject to heavy traffic are scarified once or twice during the summer months and left to be recompacted by traffic.

ROAD MATERIAL TESTS AND INSPECTION NEWS

Testing and materials engineers, chemists, and others engaged in the testing and inspection of road materials and in highway research are invited to submit articles for publication in this department of PUBLIC ROADS

WE JUDGE from the favorable comments we have received that this department of PUBLIC ROADS fills a real need of the testing engineer. It was our idea when the department was established that engineers engaged in the special field of materials testing would welcome it as a medium for the interchange of information of mutual interest in regard to their work. Acting on that idea we extended an invitation in the March issue to all testing and materials engineers and chemists to submit articles for publication. Perhaps the fact that in seven months we have not received a single contribution from anyone other than the engineers of the Bureau of Public Roads is due to the pressure of other work rather than to any lack of interest on the part of the testing engineers and chemists, but we would honestly like to see some evidence to support that view of the matter now that the busy season is past.

As an illustration of the kind of subject we would like to see discussed in this department there is the so-called strength-ratio test for concrete sand. Testing engineers are now apparently almost unanimous in the belief that this test is not a satisfactory measure of the quality of fine aggregate for concrete road construction. In view of the importance of a test for quality of fine aggregate, however, it is perfectly reasonable to assume that many laboratories have either attempted to improve the tensile-strength-ratio test or have worked on substitutes for it. The findings of the various laboratories as a result of research along this line would be of great general interest to other laboratories, even though the work done might not be considered of sufficient importance to warrant the preparation and publication of a formal article. This is just a sample of the kind of material which would be acceptable for publication in this department.

So let us repeat—Contributions are welcome. Full credit will be given for every article published. Just address them to A. T. Goldbeck, Chief, Division of Tests, U. S. Bureau of Public Roads, Washington, D. C.

FIELD INVESTIGATION OF SUBGRADE SOILS

For a number of years the bureau has been endeavoring to establish and standardize tests for determining the physical characteristics of subgrade soils in an effort to give the highway engineer definite information relative to the materials upon which he must build the highway. Up to the present date a number of tests have been developed and standardized which enable us to distinguish very readily between various types of subgrade soils from the laboratory point of view. We are now confronted, however, with the very natural question as to whether the conclusions derived from data obtained in the laboratory will hold for the soils as they exist under field conditions. In an effort to answer this question the investigations were carried into the field this spring, observation stations being established in Ohio, Iowa, and Minnesota.

As a result certain very interesting points have come up, some of the most important of which are as follows: What effect has temperature upon the capillary moisture of subgrade soils? What effect has condensation and evaporation upon the moisture content and supporting power of subgrade soils? Are subgrade failures resulting from so-called "frost boils" in the North and "mud boils" in the South attributable to the same basic cause? To what extent can drainage be relied upon to eliminate subgrade failures?

At the present time a report on the work accomplished in the field this spring is being prepared and it is expected will be available for public distribution in the near future.

THE ABRASION TEST FOR GRAVEL

The attention of the bureau has recently been called to the fact that the method for making an abrasion test for gravel, specified on page 7, U. S. Department of Agriculture Bulletin 1216, does not specify that rounded fragments only shall be used in making up a test charge. Experiments have shown that a test charge consisting of approximately 50 per cent rounded and 50 per cent crushed fragments may possibly show a percentage of wear as much as 100 per cent higher than the corresponding test sample in which all the fragments are rounded. The bureau suggests therefore that in making the abrasion test of gravel the various road material laboratories take this fact into consideration and govern their interpretation of the test results accordingly.

PORTLAND CEMENT CHECK SAMPLES DISTRIBUTED

On July 24 the bureau sent out the fifth of a series of check samples of Portland cement. This sample was sent to approximately 60 cooperating laboratories, all of which perform tests on materials used in Federal aid road construction. The bureau considers that this work is very important and urges that laboratories invited to cooperate make tests on check samples just as soon as received and not wait until the routine work has slacked up.

PORTLAND CEMENT INSPECTION

The U. S. Bureau of Standards has recently prepared in mimeographed form a summary of their practice in regard to the inspection of Portland cement. This summary goes into great detail with reference to the various steps which should be taken by the inspectors, precautions to be observed, and dangers to be guarded against. Copies of forms used by the Bureau of Standards in connection with sampling Portland cement are included. The Bureau of Standards is prepared to distribute a limited number of copies of this circular, which is entitled "Inspection of Portland Cement," by J. R. Dwyer and Roy N. Young. Requests for copies should be made direct to the Bureau of Standards.

MID-YEAR MOTOR VEHICLE REGISTRATION 15,552,077

BY G. G. CLARK, HIGHWAY ECONOMIST, U. S. BUREAU OF PUBLIC ROADS

RETURNS from the 48 States and the District of Columbia show a total registration of 15,552,077 passenger cars, taxicabs, busses, and motor trucks to July 1 of the registration year 1924. This is an increase of 2,549,650 over the registration figures of July 1, 1923, and an increase of 459,900 over the total registration for the calendar year 1923.

By geographic divisions the registration and corresponding increases and ratios of population to number of vehicles are as follows:

Geographic divisions	Vehicles	Increases over July 1, 1923	Population 1920 census	Persons per vehicle
		Per cent		
New England	1,003,023	21	7,400,909	7.3
Middle Atlantic	2,754,541	19	22,261,144	8.1
East North Central	3,966,715	19	21,475,543	5.4
West North Central	2,374,938	11	12,544,249	5.3
South Atlantic	1,427,101	23	13,990,272	9.8
East South Central	652,084	32	8,893,307	13.6
West South Central	1,237,525	21	10,242,224	8.3
Mountain	524,855	18	3,336,101	6.4
Pacific	1,611,295	25	5,566,871	3.4
Total	15,552,077	20	105,710,620	6.6

The percentage of gain in registration during the 12 months, July 1, 1923, to July 1, 1924, was largest in the East South Central States. In this group in which the gain was 32 per cent, 13.6 persons were found for each vehicle registered. The smallest percentage of gain, 11 per cent, was found in the West North Central States where one motor vehicle was registered for 5.3 persons.

A total of $199,472,682.17 was collected from the owners and operators of these vehicles for licenses and permits. Of this amount $163,011,584.29, or 82 per cent was made available for highway expenditure by or under the supervision of the respective State highway departments; the remainder was used to cover the cost of registration, to finance debts incurred for highway construction which are not payable by the highway departments, and in some States a portion was distributed to the counties.

No radical changes have been made during this period in the license fee as the total receipts from this source follow the same percentage as the rate of increase in the number of vehicles registered.

The percentage of fees made available for highway work by or under the supervision of the respective State highway departments shows a slight increase over the percentage made available during previous periods.

Full details of the registration and license fees collected by States are printed in the table on the cover of this issue. Whenever possible, passenger cars, motor trucks, taxicabs, busses, and cars for hire have been segregated in this table. However, a number of States do not make distinction between these vehicles for registration purposes and a much larger number of States do not segregate the fees received from each class. Therefore it is impossible at this time to separate in such States either the vehicles or the fees paid.

Motor-cycle registration totaling 126,850 shows a decrease of 14,671 from July 1, 1923. This decrease is in keeping with the general decrease of registration of these vehicles for several years.

GASOLINE TAXES

The tax imposed on gasoline used as motor-vehicle fuel has proved to be a source of revenue which is increasing at a rapid rate. The gross returns for July 1, 1924, when compared with similar figures for July 1, 1923, show an increase of 273 per cent.

On July 1, 1924, 35 States and the District of Columbia were collecting a tax on gasoline. During the registration year to July 1 the sum of $32,430,410.37 was collected from this source, the returns ranging from 1 cent per gallon in 8 States, 2 cents per gallon in 15 States and the District of Columbia, 2½ cents per gallon in 2 States, 3 cents per gallon in 9 States, and 4 cents per gallon in one State. Of the total collected, $20,065,581.29, or about 62 per cent, was made available for expenditure by or under the supervision of the respective State highway departments.

The total amount collected from this source for the previous registration year to July 1, 1923, was $8,669,174.03 collected in 27 States. Fifty-seven per cent of this amount was made available for road work by or under the supervision of the State highway departments. In several States parts of the proceeds of the gasoline tax are diverted to other purposes than State-supervised highway construction or maintenance and in one State the entire amount collected is diverted to other purposes.

(Continued from page 9)

That the combined license fees and gasoline taxes are not excessive in the opinion of motorists is evidenced by the fact that there is no falling off in the rate of increase in motor vehicle registration as the combined taxes are increased. Grouping the States into classes according to the amount of the combined taxes collected per vehicle-mile brings out the interesting fact that the groups in which the vehicle-mile tax was greatest reported the greatest percentage increase in motor vehicle registration. As shown by the table on page 9, the five States which collected more than 0.4 cent in combined taxes reported an average increase of 26 per cent in their motor vehicle registration. The 13 States with combined taxes ranging from 0.3 to 0.4 cent reported registration increases averaging 28 per cent. These are the two highest tax classes, and it is very interesting to note that the States in the two lower classes, with taxes from 0.2 to 0.3 cent and from 0.1 to 0.2 cent, showed average registration increases which were respectively only 22 and 23 per cent.

So long as the taxes levied upon the use of the motor vehicle do not retard the rate of increase in registration it may be safely concluded that the tax is not excessive. The above figures indicate that the motor-vehicle taxes have not been increased in any State to the point at which the law of diminishing returns becomes operative. Oregon, with its combined taxes of more than six-tenths of a cent per vehicle mile, the largest tax found in any State, reported a registration increase in 1923 of 24 per cent, which was a rate of increase equal to or greater than those reported by all but 3 of the 11 States whose combined taxes ranged from 0.1 to 0.2 cents per vehicle-mile.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

AUGUST 31,1924

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 443,164.31	$ 218,730.40	75.5	$ 15,330,056.61	$ 7,498,954.46	839.3	$ 367,310.86	$ 154,111.71	7.6	$ 1,195,018.89	Alabama
Arizona	8,338,365.41	4,297,683.98	527.8	332,420.20	174,523.00	26.2	1,999,675.90	1,212,689.70	138.6	266,950.29	163,072.20	32.7	1,657,733.22	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	122,739.41	50,850.57	4.9	6,162,288.53	2,056,909.25	328.8	2,380,491.34	1,134,998.32	110.7	1,395,296.23	Arkansas
California	12,999,075.03	5,647,148.17	533.7	1,940,703.56	1,047,037.44	58.0	11,926,181.55	6,374,801.05	436.2	1,786,901.93	841,476.45	31.9	3,182,842.89	California
Colorado	8,108,070.31	4,029,898.97	502.6	255,565.11	136,958.23	23.5	5,231,701.65	2,817,961.64	177.9	444,999.28	249,731.34	23.3	2,325,330.82	Colorado
Connecticut	3,062,872.02	1,269,568.60	73.6	199,024.56	98,423.00	4.9	2,851,926.15	936,604.04	47.4	410,228.33	109,200.00	6.7	967,409.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	462,969.09	197,925.82	13.9	608,205.95	214,911.10	13.9	588,061.54	233,370.00	15.6	85,708.25	Delaware
Florida	961,134.07	461,470.92	48.3				9,190,174.92	4,506,551.34	259.4	398,642.71	171,123.94	10.2	1,147,740.90	Florida
Georgia	17,167,373.32	7,955,805.20	1214.2	479,165.95	238,182.21	40.6	9,303,989.84	4,592,560.37	718.8	2,021,929.52	966,076.73	137.2	697,272.49	Georgia
Idaho	8,181,697.92	4,032,395.52	506.9	197,306.67	113,420.29	18.7	1,761,847.73	1,021,213.39	123.4	851,082.12	547,590.29	48.3	903,092.52	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	493,620.16	244,171.81	14.6	16,450,654.64	8,174,734.64	558.4	153,358.60	76,679.30	6.1	2,661,359.92	Illinois
Indiana	7,577,444.16	3,655,540.97	225.7	461,440.45	226,266.09	19.2	15,692,278.63	7,651,303.31	510.0	1,001,958.16	481,030.23	30.6	2,298,251.40	Indiana
Iowa	23,195,778.19	9,237,031.86	1682.9	98,400.80	43,700.00	2.6	8,721,632.22	4,054,399.47	641.4	356,429.80	173,200.00	34.3	1,927,805.67	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	3,276,833.27	1,166,738.82	94.3	15,550,532.64	6,153,206.97	597.5	1,235,074.78	611,580.79	101.6	1,324,595.62	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4				7,691,317.09	3,713,743.94	329.3	358,008.82	179,004.40	23.6	1,865,043.38	Kentucky
Louisiana	8,438,463.19	3,636,143.36	661.2	113,769.13	54,557.86	7.8	5,657,879.79	2,804,437.19	364.6	152,006.67	76,003.33	12.3	694,300.26	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	260,943.30	129,792.71	10.0	1,505,435.72	748,285.60	56.6	222,227.50	111,113.74	12.0	800,844.57	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	79,449.48	39,724.24	3.7	2,301,910.45	1,007,450.21	72.9	735,978.43	270,647.31	21.7	117,806.46	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	88,434.91	28,760.00	1.4	5,930,451.96	1,970,232.69	101.9	407,133.25	126,791.38	5.9	1,688,268.81	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5				13,928,561.21	6,708,729.97	494.3	45,171.00	22,585.50	1.4	3,087,844.40	Michigan
Minnesota	24,037,561.24	9,885,843.07	2292.0	108,570.20	39,500.00	12.2	9,902,351.38	4,590,829.97	815.6	1,045,190.59	519,800.00	148.1	292,445.96	Minnesota
Mississippi	7,898,193.99	3,828,941.39	655.0	61,901.51	30,950.75	9.6	7,837,647.89	3,910,833.44	480.5	1,007,528.02	497,508.48	47.9	1,262,978.94	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.5	35,842.36	17,921.18	0.1	16,929,374.31	8,087,053.65	721.5	3,956,018.91	1,905,870.60	208.4	2,683,443.39	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	109,869.05	97,529.84	12.4	2,876,282.84	2,107,672.46	268.2	1,063,490.31	604,120.51	120.3	3,782,759.07	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	200,902.81	100,401.40	22.6	7,221,153.76	3,510,057.71	733.9	1,014,509.69	506,904.60	112.4	3,618,990.70	Nebraska
Nevada	3,460,245.52	1,953,624.99	225.6	587,270.26	511,310.71	43.3	4,662,427.47	3,903,926.93	397.8	2,547.71	2,222.11		619,236.27	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3				1,252,298.78	586,791.62	40.7	233,335.90	112,525.58	8.0	247,779.22	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	298,751.09	111,360.00	5.6	9,724,981.71	2,795,603.75	73.4	796,997.14	205,677.49	10.7	815,074.27	New Jersey
New Mexico	5,306,286.45	2,758,849.68	714.3	62,583.77	38,142.25	4.3	7,016,229.26	4,589,987.38	709.5	311,857.56	191,792.38	22.0	1,010,560.31	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	391,274.42	153,023.98	12.5	27,029,766.96	10,351,676.72	608.3	7,213,038.00	2,213,920.00	140.7	5,731,682.96	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	39,212.36	16,990.22	7.6	11,116,708.27	4,156,350.61	301.0	1,505,316.49	709,351.48	43.6	1,734,901.03	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1597.9	139,674.74	69,724.43	25.0	3,634,215.34	1,800,155.68	577.7	259,135.37	110,437.00	20.8	1,964,933.47	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	742,323.36	294,754.13	19.7	13,028,842.05	4,927,429.79	353.7	2,641,527.60	896,444.89	73.0	2,141,617.20	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	738,661.44	349,064.35	32.5	8,183,798.49	3,966,056.68	355.0	2,282,876.56	991,165.07	114.2	1,342,564.87	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	513,232.52	303,539.04	48.5	2,419,116.36	1,397,424.72	127.1	916,370.40	560,238.86	54.1	425,862.59	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	415,284.23	176,961.99	9.7	10,296,779.06	5,754,526.25	344.3	4,675,257.92	1,226,136.25	80.9	3,329,296.72	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	157,932.08	62,600.00	3.1	1,243,899.16	438,306.80	25.2				652,906.24	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	454,462.14	216,444.36	90.9	5,984,408.28	2,478,502.93	438.1	628,722.95	235,728.45	71.9	632,825.04	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	551,979.03	306,772.84	85.7	7,136,700.83	3,587,283.47	990.4	817,598.02	378,330.33	74.7	201,657.09	South Dakota
Tennessee	6,805,683.35	3,313,936.07	259.6	854,869.74	427,434.87	24.0	12,936,444.00	6,034,273.41	409.3	3,158,756.70	1,364,493.04	129.6	884,499.61	Tennessee
Texas	42,341,998.56	16,150,624.91	3122.8	1,140,024.71	538,466.86	121.5	24,917,369.29	9,900,561.39	1517.1	4,021,240.94	1,515,009.95	266.7	3,579,549.89	Texas
Utah	3,304,423.76	1,895,805.92	219.0	719,066.79	390,456.24	91.2	3,658,519.93	2,411,856.29	212.9	1,161,249.33	800,886.74	74.7	617,467.91	Utah
Vermont	1,922,114.16	942,769.12	74.4				1,794,475.68	863,325.61	45.8	255,272.32	118,784.62	6.5	609,099.65	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	141,742.05	70,871.02	4.6	9,029,422.39	4,175,782.70	308.7	1,659,955.26	567,301.18	58.0	997,215.67	Virginia
Washington	11,384,615.67	5,290,895.45	457.0				2,951,003.81	1,306,000.00	121.6	1,341,215.00	621,300.00	53.6	668,482.55	Washington
West Virginia	5,489,747.95	2,365,041.53	255.6	898,083.14	402,593.56	24.7	4,428,254.79	1,998,238.67	155.6	528,260.11	232,245.33	18.5	756,022.91	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1325.3	268,053.95	134,026.97	9.2	4,301,465.29	2,141,404.69	191.6	572,540.18	274,399.01	17.5	3,687,586.76	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	35,927.39	23,179.25	1.1	4,672,794.59	2,949,235.74	353.9	169,412.58	94,122.00	4.8	543,715.31	Wyoming
HAWAII													365,625.00	HAWAII
TOTALS	$ 549,655,391.27	$ 237,852,399.82	32452.9	$ 18,970,344.39	$ 9,082,572.73	1140.3	$ 393,768,925.00	$ 178,929,788.19	18488.7	$ 57,421,966.39	$ 24,146,102.90	2655.2	$ 75,114,136.36	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 56,371,163.31 Federal aid $ 26,175,885.42 Miles 2795.2

MOTOR VEHICLE REGISTRATION AND REVENUES, JULY 1, 1924

State	Grand total motor cars	Private passenger cars	Motor trucks	Taxis, busses, and cars for hire [1]	Motor cycles	Total gross receipts: From registration fees, licenses, and permits	Total gross receipts: From gasoline tax [2]	Amount applicable to highway work by or under supervision of State highway department: From registration fees, licenses, and permits	Amount applicable to highway work by or under supervision of State highway department: From gasoline tax	Amount of registration fees paid: Passenger cars [3]	Amount of registration fees paid: Motor trucks [3]	Total registration to July 1, 1923	Increase July 1, 1924, over July 1, 1923	State
													Per cent	
Alabama	154,234	133,216	18,225	2,793	330	$1,554,979.65	$737,572.58	$1,219,104.05				97,788	57	Alabama.
Arizona	50,409	43,616	6,793		286	313,668.00	334,768.59	313,668.00	$167,384.30			41,630	21	Arizona.
Arkansas	116,700	103,600	13,100		219	2,299,828.31	980,804.36	2,299,828.31	980,804.36			97,147	20	Arkansas.
California	1,184,015	1,017,556	166,459		10,615	5,794,078.41	6,000,000.00	[4] 2,800,000.00	3,000,000.00	$3,225,333.15	$2,163,781.81	933,808	27	California.
Colorado	186,926	173,856	13,070		1,723	1,143,289.99	748,404.48	[4] 500,000.00	374,202.24	925,025.83	161,539.46	159,001	17	Colorado.
Connecticut	188,344	156,621	29,142	2,581	3,299	4,545,130.30	[5] 348,585.35	4,545,130.30	[5] 348,585.35	2,575,771.74	1,008,590.40	152,396	24	Connecticut.
Delaware	31,300	26,000	5,300		271	544,628.00	120,385.22	544,628.00	120,385.22	307,238.00	112,281.00	26,300	19	Delaware.
District of Columbia	72,954	63,405	8,479	1,070	1,431	299,265.00	[6] 83,391.71		[6] 83,391.71			62,777	16	District of Columbia.
Florida	164,112	132,807	28,616	2,689	597	1,515,667.50	1,837,406.34	[4] 1,050,000.00	1,224,338.42			135,896	21	Florida.
Georgia	177,030	154,280	22,650	100	670	2,315,603.55	1,648,252.24	2,252,844.45	549,417.41	1,891,938.20	375,236.40	149,900	19	Georgia.
Idaho	62,175	55,803	6,372		519	1,223,097.45	225,532.64	305,774.36	225,532.64	1,018,673.70	176,147.75	55,631	12	Idaho.
Illinois	986,480	862,452	124,028		5,387	10,696,508.51		10,696,508.51		7,708,796.28	2,389,564.47	833,920	18	Illinois.
Indiana	581,899	508,831	72,744	324	3,794	4,813,400.00	2,107,601.08	3,673,812.20	2,064,302.11	3,636,219.00	965,784.00	486,848	20	Indiana.
Iowa	569,415	531,719	37,696		2,336	8,531,030.04		7,823,000.00				517,228	10	Iowa.
Kansas	360,796	327,004	33,792		1,195	[4] 3,835,370.00		[4] 3,642,851.00				332,068	9	Kansas
Kentucky	203,028	182,599	20,429		568	3,007,968.58	392,020.65	2,971,122.50	392,020.65			167,336	21	Kentucky
Louisiana	150,000	125,000	25,000		475	2,480,706.00	427,583.54	2,480,706.00	427,583.54	2,001,000.00	431,241.00	110,000	36	Louisiana.
Maine	110,282	91,803	16,004	2,475	1,124	1,663,263.43	165,166.40	1,579,326.30	165,075.75			96,333	15	Maine.
Maryland	178,153	165,636	10,082	2,435	2,827	1,871,171.45	563,944.46	1,201,963.46	550,095.24	1,249,288.49	237,284.45	163,654	9	Maryland
Massachusetts	508,997	433,695	75,302		8,438	6,668,069.00		6,668,069.00		4,555,082.50	1,138,202.50	420,822	21	Massachusetts
Michigan	764,423	691,707	72,716		3,046	10,950,392.93		5,143,671.24		8,886,436.43	1,286,753.85	624,590	22	Michigan.
Minnesota	462,777	429,308	33,150	319	2,754	8,012,922.65		8,012,922.65		6,953,684.67	839,426.76	405,225	14	Minnesota.
Mississippi	112,099	100,889	11,210		85	1,127,948.46	265,711.00	586,140.74	106,284.40			85,476	32	Mississippi
Missouri	477,056	430,621	46,435		1,706	3,928,326.02		3,928,326.02				411,525	16	Missouri.
Montana	69,100	61,100	8,000		223	698,755.50	[4] 125,525.40		[4] 25,105.08	579,494.50	92,785.50	63,950	8	Montana.
Nebraska	273,236	246,415	26,821		948	3,336,541.75		2,414,800.00		2,746,374.96	521,256.93	243,347	12	Nebraska.
Nevada	15,961	15,961	(7)		95	167,010.50	46,382.48					12,492	27	Nevada.
New Hampshire	64,373	64,373	(7)		1,431	923,063.21	204,399.65	852,064.96	204,399.65	756,129.82		52,584	22	New Hampshire
New Jersey	435,894	338,133	[5] 88,096	9,665	6,398	8,079,325.75		7,379,006.68		3,318,670.20	2,523,248.00	360,577	20	New Jersey.
New Mexico	34,951	33,499	108	1,344	182	344,813.41	84,993.90	327,572.74	80,744.21	294,227.19	2,467.63	24,937	40	Mew Mexico.
New York	1,238,362	996,392	203,877	33,093	16,725	20,197,362.41		20,197,362.41		12,808,938.41	5,748,547.00	1,025,718	20	New York.
North Carolina	285,546	260,667	24,879		1,488	3,975,658.37	2,024,421.68	3,578,092.55	1,821,979.51			209,400	36	North Carolina.
North Dakota	104,845	104,845	(7)		425	706,628.85	94,573.94	706,628.85				99,000	6	North Dakota.
Ohio	1,160,000	999,300	160,700		13,590	10,509,916.50		5,254,958.25				965,000	20	Ohio.
Oklahoma	308,906	298,906	10,000		850	2,894,682.63	1,042,700.00		447,000.00			240,000	29	Oklahoma.
Oregon	161,739	149,647	12,092		2,235	4,336,682.54	1,089,472.94	[4] 3,192,510.00	1,045,108.47	3,610,614.50	622,287.29	133,657	21	Oregon.
Pennsylvania	1,085,285	924,274	154,830	6,181	14,367	19,562,394.31	3,524,018.90	19,562,394.31		9,582,379.80	4,607,396.70	922,062	18	Pennsylvania.
Rhode Island	78,413	62,887	14,268	1,258	1,122	1,312,397.85		1,260,183.30		804,363.62	310,564.91	64,457	22	Rhode Island.
South Carolina	137,891	125,215	12,676		382	1,069,751.29	992,726.52	855,801.02	330,848.24	869,506.95	177,419.52	102,127	35	South Carolina.
South Dakota	126,813	117,812	9,001		260	1,952,200.00	447,554.00	1,366,540.00	393,847.00	1,737,720.00	192,980.00	120,021	6	South Dakota.
Tennessee	182,723	164,119	18,604		601	2,391,849.64	724,391.01	1,195,924.82	724,391.01			144,902	26	Tennessee
Texas	661,919	611,047	47,906	2,966	2,135	9,452,542.45	1,762,720.65	6,536,037.52	1,322,040.49			571,981	16	Texas.
Utah	67,133	58,784	8,349		600	442,920.43	286,172.77	[4] 400,000.00	[4] 275,000.00			50,798	32	Utah.
Vermont	52,614	49,053	3,561		639	1,186,230.61	[5] 48,230.03	[8] 810,000.00	48,230.03	917,210.46	92,196.00	44,613	18	Vermont.
Virginia	225,343	197,052	28,291		1,350	3,371,208.20	[5] 1,132,534.80		755,023.20	2,696,445.34	452,377.41	184,961	22	Virginia.
Washington	265,541	227,015	36,423	2,103	2,457	4,242,122.95	1,290,785.08	[4] 4,000,000.00	1,290,785.08	3,046,155.20	882,648.70	220,667	20	Washington.
West Virginia	154,772	133,572	17,650	3,550	1,094	2,552,544.35	445,032.36	2,552,544.35	445,032.36	1,751,659.88	405,797.98	124,341	25	West Virginia.
Wisconsin	473,913	429,634	44,279		3,348	6,226,933.43		5,926,933.43		5,065,570.50	977,320.00	418,536	13	Wisconsin.
Wyoming	38,200	34,000	4,200		210	402,832.01	76,643.62	402,832.01	76,643.62			35,000	9	Wyoming.
Total	15,552,077	13,645,726	1,831,405	74,946	126,850	199,472,682.17	32,430,410.37	163,011,584.29	20,065,581.29	95,519,949.32	28,895,127.42	13,002,427	20	Total.

[1] Where no data are given these vehicles are not registered as a separate class but are included with passenger cars or motor trucks.
[2] Where no data are given, gasoline tax laws are not in force.
[3] Where no data are given the States do not segregate the fees for these classes of vehicles
[4] Estimated; final figures not available
[5] To May 31 only.
[6] Gas tax law effective May 24, 1924.
[7] Included with passenger cars.
[8] Already appropriated by legislature; more may be available later.

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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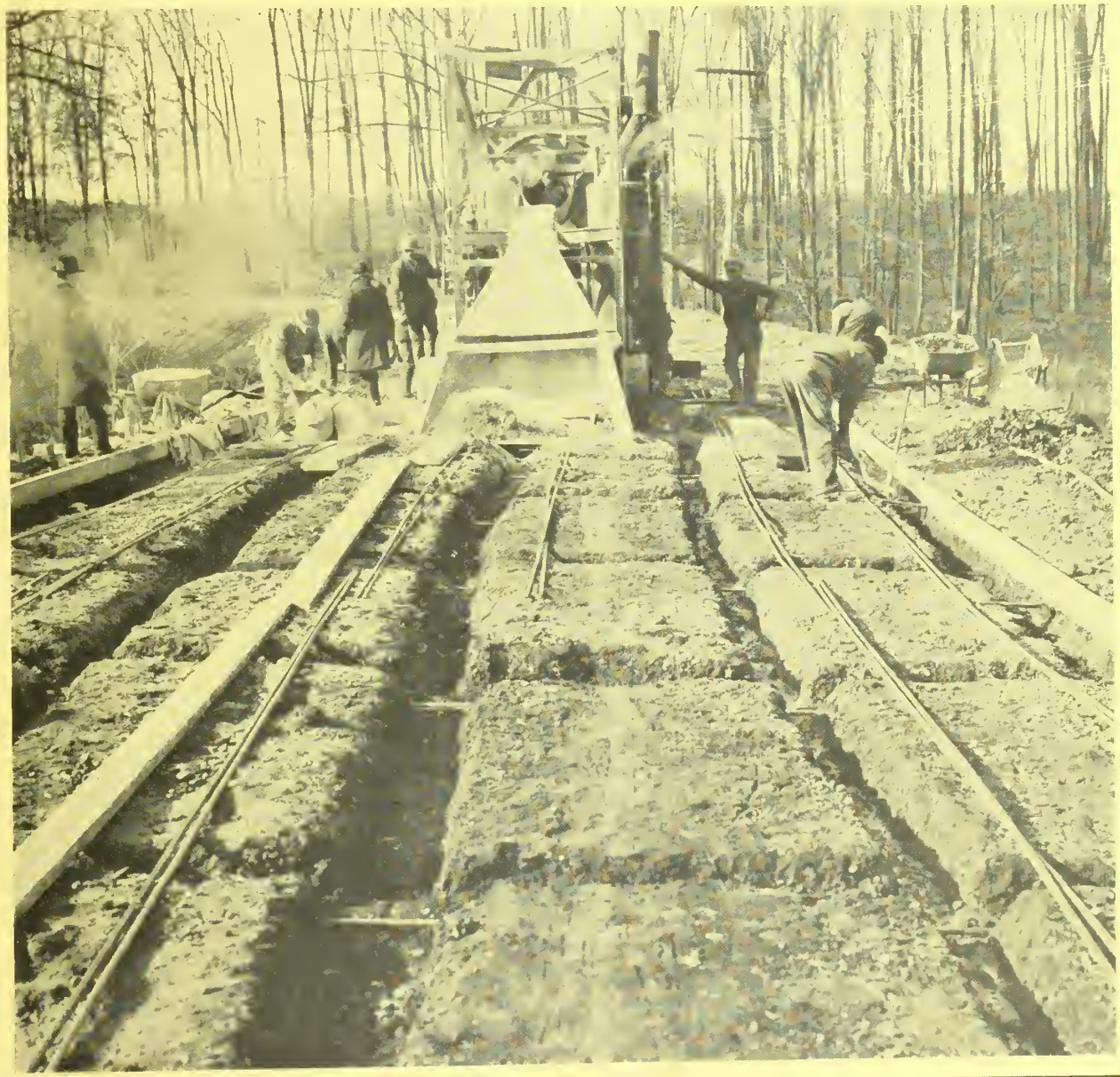

THE SUBGRADE FOR SECTION 30, COLUMBIA PIKE EXPERIMENTAL ROAD

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PUBLIC ROADS

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REINFORCING AND THE SUBGRADE AS FACTORS IN THE DESIGN OF CONCRETE PAVEMENTS

A STUDY OF EXPERIMENTAL SECTIONS OF THE COLUMBIA PIKE

By J. T. PAULS, Associate Highway Engineer, United States Bureau of Public Roads

OBSERVATIONS of the Columbia Pike, an experimental road constructed by the United States Bureau of Public Roads near Arlington, Va., after two and a half years' service under traffic have made possible a number of conclusions with regard to the relation between the cracking of concrete roads and the character of the subgrade and steel reinforcing. The observations also reveal a number of interesting facts with reference to the water-holding properties of the soils composing the subgrades and the relation between these properties and resulting changes in volume.

It is found that subgrade materials with a large percentage of clay not only attain a high moisture content during the wet season but retain a high content during the dry season. Materials of this character subjected to the laboratory test for moisture equivalent will be found to have a high moisture equivalent value. Subgrades having a large percentage of sand do not attain high moisture content. It is also found that subgrade materials composed largely of very fine sand have high capillarity and that under this condition free water will very often be found between the pavement and the subgrade.

Subgrades composed largely of clay swell and contract as moisture is added or taken away. The effect of this in the case of swelling is to lift the pavement at the edges and on contraction to take away the support at the edges. The result is that the slab, acting in one case as a simple beam and in the other as a cantilever, is broken at the center by traffic. The conclusion is drawn from the observations that subgrades that show as much as 10 per cent volume change, by laboratory test on an entire sample including coarse material, should be covered with a layer of coarse granular material, and a pavement laid on a subgrade of this character should have a longitudinal joint at the center. Longitudinal cracks in a pavement indicate an unstable subgrade either as to supporting value or movement caused by moisture changes. Adding a granular material to such a subgrade increases its supporting value and modifies the effect of any volume change. Pavements on this type of subgrade should be designed with a center joint.

The observations made indicate that plain concrete slabs will crack transversely because of temperature and moisture changes at intervals of from 40 to 60 feet. Smooth subgrade surfaces increase the distances between cracks, but the thickness of the concrete does not affect the spacing of the contraction cracks.

Judging by the experience with the experimental sections, pavements reinforced longitudinally will develop transverse contraction cracks, the number, spacing, and size of which will be controlled by a number of factors. If the steel reinforcing is not continuous but is separated by joints, it is to be expected that no cracks will form less than 30 feet from any joint, and by a suitable relation of the percentage of steel to the length over which the steel is made continuous the distance may be increased to 60 feet. The position of the cracks will be influenced by the strength of the concrete and the roughness of the subgrade as well as by the percentage and continuous length of the steel. If the spacing of the joints is less than twice the distance in which a crack would form, contraction cracking may be entirely prevented. If the distance between joints is extended, a crack may be anticipated at a distance of 30 to 60 feet from each joint and the area between these cracks may be expected to crack at relatively short intervals. With a high percentage of continuous steel relatively fine, closely spaced cracks may be looked for; with a low percentage breaks in the steel may be expected to permit wider cracks to form at considerable intervals. Mesh reinforcing in amounts as used in these tests is likely to break at intervals and permit open cracks to form.

Attention should be called to the possible danger of the use of too high a percentage of longitudinal steel, for under such conditions numerous fine transverse cracks will develop and there is the possibility that the narrow transverse beams thus formed will crack under traffic.

From the results obtained on sections reinforced longitudinally it would appear that the practice of omitting contraction joints in pavements of this type is questionable. It would appear that where longitudinal steel is used the design should provide for contraction joints from 50 to 100 feet apart with the steel designed to prevent intermediate contraction cracks from forming. Another method that would probably be more satisfactory to the contractor, but which might be subject to other objections, would be to make the concrete continuous and to break the steel one-half inch or 1 inch at intervals where it is desired that contraction cracks shall form.

These conclusions will, no doubt, be modified in certain respects and in others made more conclusive as time permits the action of the various factors to become more effective. The results obtained thus far show conclusively the great importance of subgrade investigations in connection with the design of a pavement. Since the character of the subgrade may vary widely in a comparatively short section of road, it seems logical that the design should be modified accordingly so as best to meet the particular subgrade conditions existing, or that the subgrade should be so corrected as to make it suitable for a uniform design of pavement.

THE CHARACTER OF EXPERIMENTAL SECTIONS

The Columbia Pike includes 32 experimental concrete surfaces of an average length of 200 feet, varying in

12360—24†—1

such features as the thickness, reinforcing, and cross section of the slabs. The nine types of cross section used are shown in Figure 1, the principal dimensions and characteristics of each in Table 1. All sections were made of Potomac River aggregate and cement in the proportion of 1 : 2 : 3.

Generally speaking, the subgrade was very stable. The old macadam construction, loosened, reshaped and compacted, served as the base of some of the sections; others, on account of changes in grade, were laid directly on the earth, with the exception of two sections, Nos. 9 and 28, which were laid on a base constructed of cinders. Sections 30, 31, and 32 were built on a heavy fill in new location. The nature of the soil underlying each of the sections is represented in detail by the analyses of samples taken every 50 feet at the time of construction. These analyses are reported in Table 2.

Capped pipes were embedded in the concrete at intervals when the concrete was placed, and at various times since the pavements were completed samples of the subgrade material have been extracted through these pipes for determination of the moisture content. The results of these determinations are recorded in Table 3.

Besides making these moisture determinations, the condition of the pavement has been observed and a record has been kept of the cracks that have formed. The results of these observations are recorded in Figures 2, 3, and 4, which show, in addition to the record of cracks, the average, maximum, and minimum moisture determinations and the analyses of the subgrade materials.

The general condition of the experimental sections at this time is good. Certain sections, such as 1 and 29 between stations 101 and 103 and stations 105 and 108, which have cracked badly, may give trouble later. Section 9, a 4-inch plain concrete section on a cinder base, is badly cracked over a small area in the vicinity of a joint. The slipping of the concrete at this joint, due to the fact that the joint had been placed off the vertical at the time of construction, may be held responsible for the failure of this section. Other than in these cases the experimental sections are in such condition as to require only the regular periodic joint and crack repair customary on this type of road.

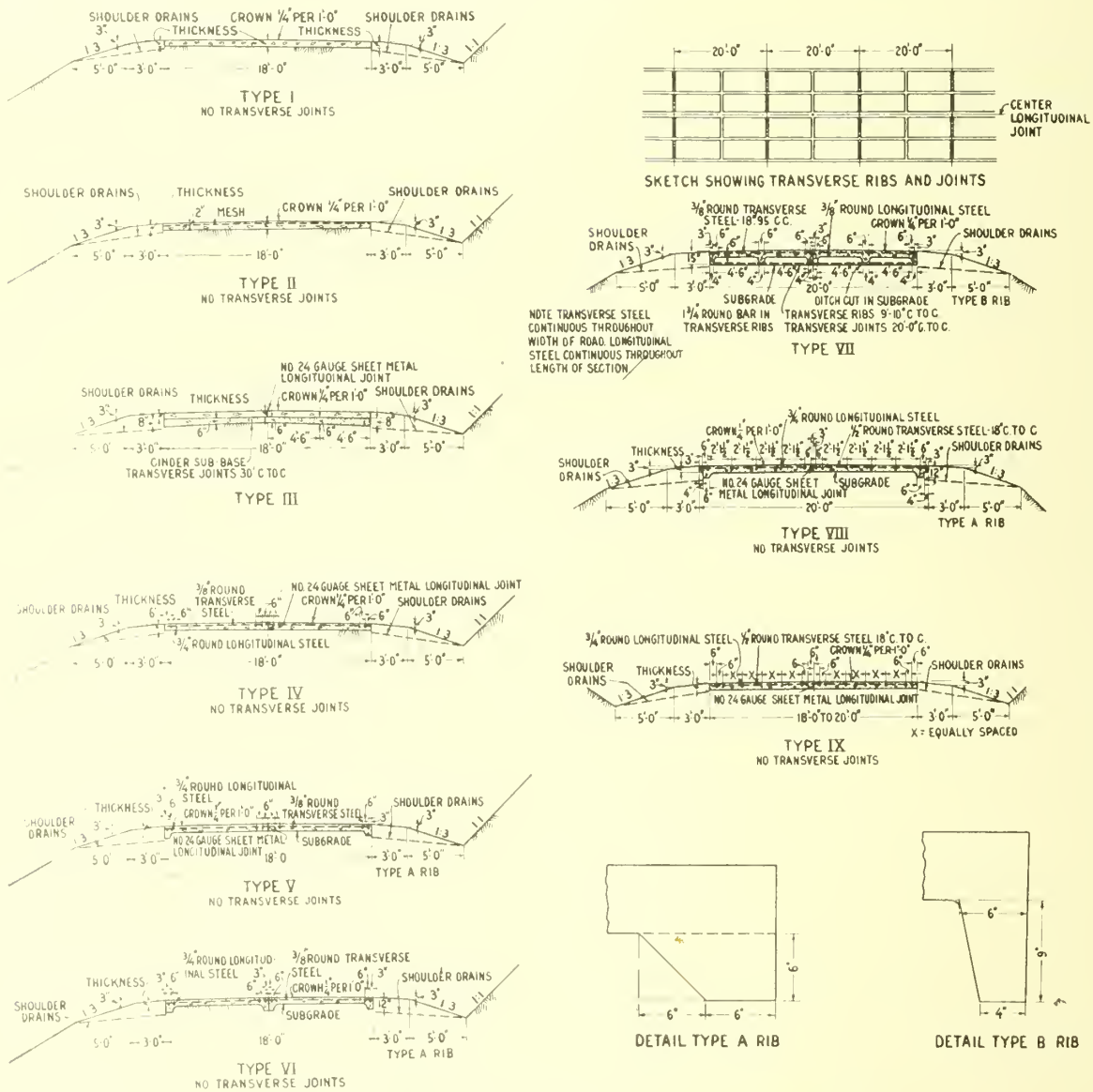


FIG. 1.—The types of cross section and characteristics of sections of the Columbia Pike

TABLE 1.—*Dimensions and characteristics of experimental sections*

Section No.	Station	Type	Thickness	Reinforcement		Joints		Concrete ribs		Constructed base
				Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	Transverse	
			Inches							
1	27+55 to 29+55	I	6-7	None	None	None	None	None	None	None.
2	29+55 to 31+55	II	6	None	None	None	None	None	None	None.
3	31+55 to 33+55	II	7	None	None	None	None	None	None	None.
4	33+55 to 35+55	II	8	None	None	None	None	None	None	None.
5	35+55 to 39+55	I	6-7	None	None	None	None	None	None	None.
6	39+55 to 41+55	II	6	25 pounds per 100 square foot mesh		None	None	None	None	None.
7	41+55 to 43+55	II	6	50 pounds per 100 square foot mesh		None	None	None	None	None.
8	43+55 to 45+51	I	7 8	None	None	None	None	None	None	None.
9	45+51 to 47+55	III	4	None	None	Center	30 feet center to center.	None	None	6–8 inch cinder.
10	47+55 to 49+55	IV	6	None	None	do	None	None	None	None.
11	49+55 to 51+55	I	8 9	None	None	None	None	None	None	None.
12	51+55 to 53+55	IV	7	None	None	Center	None	None	None	None.
13	53+55 to 55+55	IV	8	None	None	do	None	None	None	None.
14	55+55 to 59+55	I	6 7	None	None	None	None	None	None	None.
15	59+55 to 61+55	IV	6	8 ½-inch round rods.	⅝-inch round rods, 12 inches center to center.	Center	None	None	None	None.
16	61+55 to 63+55.5	IV	6	do	⅜-inch round rods, 18 inches center to center.	do	None	None	None	None.
17	63+55.5 to 65+56.5	IV	6	do	⅜-inch round rods, 24 inches center to center.	do	None	None	None	None.
18	65+56.5 to 67+56.5	IV	6	4 ½-inch round rods.	None	do	None	None	None	None.
19	67+56.5 to 69+56.5	IV	6	4 ¾-inch round rods.	None	do	None	None	None	None.
20	69+56.5 to 71+56.5	IV	6	4 1-inch round rods.	None	do	None	None	None	None.
21	71+56.5 to 73+56.5	V	6	None	None	do	None	2 Type A	None	None.
22	73+56.5 to 75+56.5	V	8	None	None	do	None	do	None	None.
23	75+56.5 to 77+56.5	V	6	8 ¾-inch round rods.	None	do	None	do	None	None.
24	77+56.5 to 79+57	V	6	do	⅜-inch round rods, 18 inches center to center.	do	None	do	None	None.
25	79+57 to 81+50	VI	6	None	None	do	None	4 Type A	None	None.
26	81+50 to 83+50	VI	6	8 ¾-inch round rods	None	do	None	do	None	None.
27	83+50 to 85+50	VI	6	do	⅜-inch round rods, 18 inches center to center.	do	None	do	None	None.
28	85+50 to 87+00	III	6	None	None	do	30 feet center to center.	None	None	6–8 inch cinder
29	87+00 to 108+00	I	6 7	None	None	None	None	None	None	None.
30	108+00 to 110+00	VII	6	Shown on plan, Fig. 1.		Center	20 feet center to center.	6 type B	9 feet 10 inches center to center.	None.
31	110+00 to 112+00	VIII	6	14 ¾-inch round rods.	½-inch round rods, 18 inches center to center.	do	None	2 type A	None	None.
32	112+00 to 115+50	IX	6	do	do	do	None	None	None	None.

CHARACTERISTICS OF STEEL REINFORCING BARS

The steel reinforcing bars used in the various reinforced sections were deformed round bars made of billet steel for concrete reinforcement, intermediate grade, conforming to A. S. T. M. standard specification A15–14. The mesh reinforcement consisted of main members held together by secondary members at right angles to and twisted about the main members.

The 1-inch rods used were not tested. Tests of the three-eighths, one-half, and three-fourths inch rods revealed the characteristics shown in the following table:

Size of rod	Area of cross section	Ultimate strength	Elongation in 8 inches	Yield point
	Square inches	*Pounds per square inch*	*Per cent*	*Pounds per square inch*
⅜-inch	0.11	72,750	22.6	49,100
½-inch	.20	75,550	22.6	47,750
¾-inch	.44	77,950	25.0	45,500

THE AMOUNT AND EFFECT OF SUBGRADE MOISTURE

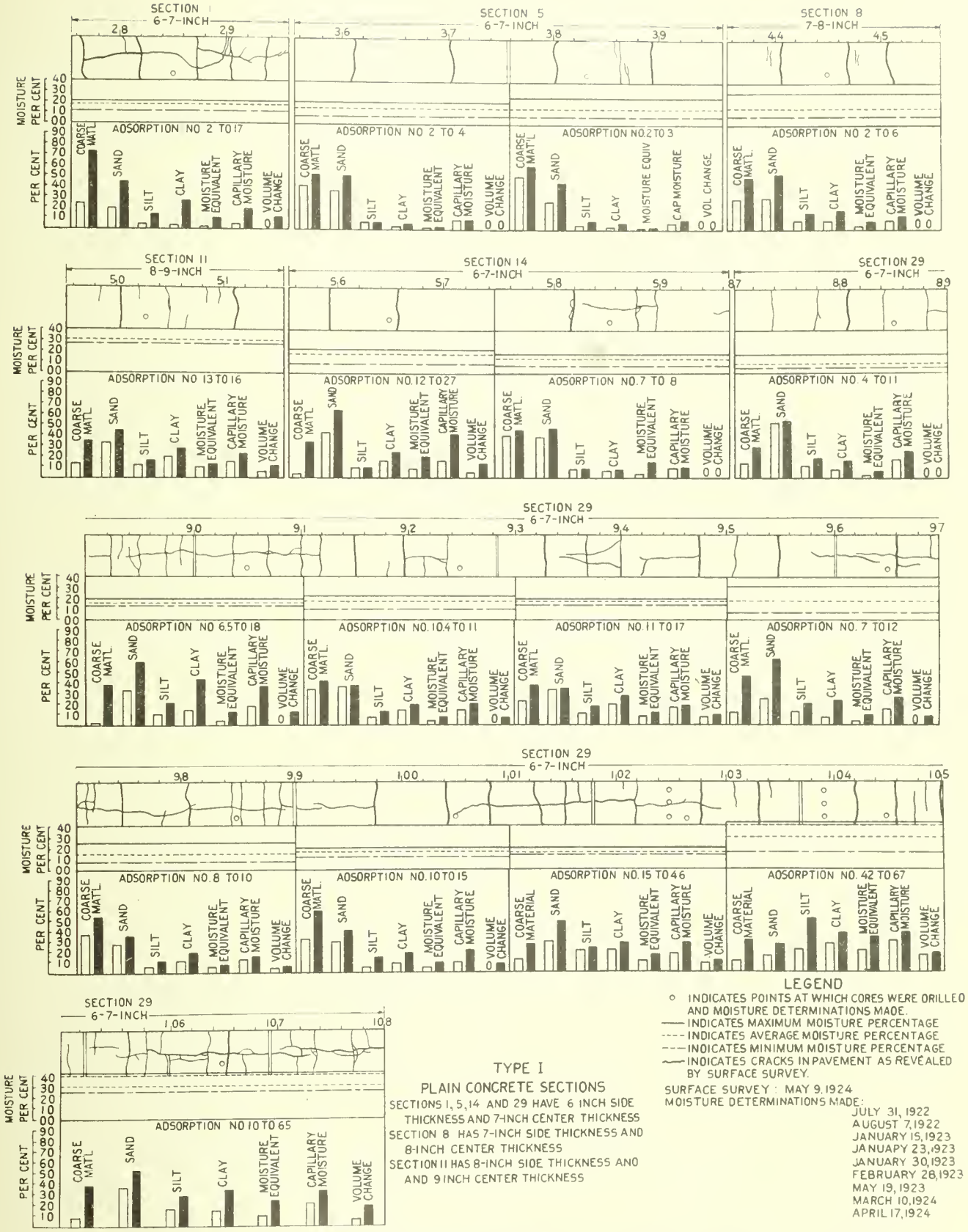
Marked variations were observed in the moisture content of the subgrade under the various sections and in the length of time that high percentages of moisture were retained after the advent of dry weather. The highest percentage of moisture recorded was 43 per cent between stations 105 and 108, while the lowest percentage observed at the same time was 8.2 per cent in section 16. The highest moisture content found during a dry period was 28 per cent in section 11 and the lowest observed at the same time was 5.5 per cent in section 29. These figures show that the subgrade in some locations may be almost free of moisture while at the same time the subgrades in other sections may be almost saturated. This occurs in portions of the subgrade where conditions other than the subgrade materials are the same. In view of the change in the supporting value of certain subgrade materials on addition of moisture, it would seem very important to determine how those subgrades compare in their supporting value with other materials that do not attain such high moisture content.

The subgrade materials which show high moisture content are found to be those materials which have a high percentage of clay. Sections 4, 10, 11, 32, 31, 30, and section 29, between stations 89–91, 93–99, and 103–108, have subgrades of this type. Moisture determinations on these subgrades not only show high content during the wet season but also show a large amount retained during the dry season.

TABLE 2.—*Analyses of subgrade materials—Samples obtained from the top 6-inch layer of finished grade along the center line of the pavement*

Section No.	Type	Station	Mechanical analysis[1]										Moisture equivalent	Capillary moisture	Volume change caused by capillary moisture	Adsorption number
			Coarse material		Sand				Silt	Clay						
			Above ¼-inch	¼-inch–10 mesh	10–20 mesh	20–60 mesh	60–100 mesh	100–200 mesh								
			Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	Per cent	
1	1	28+00	18.5	4.8	4.9	20.2	6.4	6.4	13.4	25.5	9.2	18.0	10.3	17.0		
		28+50	67.9	5.8	3.4	10.3	2.7	2.8	4.0	3.0	1.6	4.6	0.0	2.0		
		29+00	35.6	29.8	7.2	6.8	1.8	2.8	6.4	9.6	4.7	9.2	4.1	12.0		
		29+50	25.3	13.9	2.8	23.5	11.4	6.0	4.9	12.1	6.3	15.9	4.3	10.0		
2	II	30+00	6.8	11.0	5.7	24.6	9.8	9.0	14.7	18.4	7.6	16.9	5.4	9.0		
		30+50	9.4	11.2	19.8	41.0	1.9	1.6	4.5	10.4	5.4	11.1	0.0	3.0		
		31+50	51.6	20.9	2.8	9.8	2.2	3.0	4.5	5.2	4.3	10.3	2.5	9.0		
3	II	32+50	56.3	9.2	4.7	11.4	4.3	4.2	6.4	3.5	1.3	5.0	0.0	3.0		
		33+00	38.2	21.1	3.2	15.5	4.6	5.8	6.8	4.7	2.1	7.7	0.0	4.0		
4	II	33+50	36.0	9.9	6.0	23.3	4.5	4.0	6.8	9.5	4.6	10.6	0.0	7.0		
		34+00	11.0	8.6	11.5	29.8	7.1	6.2	12.9	12.8	6.7	17.9	0.0	9.0		
		34+50	6.0	8.3	4.5	23.3	7.3	10.5	22.5	17.5	10.7	18.2	0.0	10.0		
		35+00	20.8	4.4	4.3	18.0	7.8	8.8	21.5	14.4	8.1	17.6	0.0	11.0		
	I	36+00	28.2	11.9	9.8	23.4	8.5	8.7	6.7	2.8	1.4	8.9	0.0	2.0		
		37+00	46.6	5.3	5.2	20.7	5.3	4.9	6.7	5.2	2.4	8.9	0.0	4.0		
5	I	38+50	42.6	7.6	6.9	24.6	6.6	6.1	3.9	1.8	0.9	8.5	0.0	2.0		
		39+50	53.6	7.1	4.2	12.4	4.6	4.4	7.7	6.1	2.4	5.7	0.0	3.0		
6	II	40+00	35.4	9.8	5.1	17.5	5.5	6.7	10.8	9.4	3.7	9.5	0.0	6.5		
		40+50	50.5	5.2	2.5	12.1	4.9	4.1	9.9	10.8	4.1	7.5	0.0	5.5		
		41+00	23.8	6.2	4.6	22.6	7.8	9.7	14.3	11.1	5.6	16.7	0.0	9.0		
		41+50	28.8	9.1	5.2	20.7	6.3	7.5	11.9	10.5	4.2	10.5	0.0	6.0		
7	II	43+50	37.3	9.1	5.3	16.5	4.4	3.4	5.7	18.3	7.7	13.1	5.1	11.0		
8	II	44+00	35.1	4.9	3.0	14.3	5.6	6.1	14.3	16.7	6.8	10.3	0.0	4.0		
		44+50	22.0	6.2	4.8	27.1	9.6	10.6	12.3	7.4	2.7	9.8	0.0	2.0		
		45+00	30.6	5.3	4.4	20.9	8.4	6.8	11.0	12.5	4.2	11.5	0.0	4.0		
		45+50	39.4	9.1	5.5	18.7	5.1	6.2	8.4	7.7	3.1	7.8	0.0	6.0		
9	IV	48+00	30.0	6.5	2.9	13.5	5.1	5.7	11.5	24.7	9.8	14.9	8.3	10.0		
		48+50	16.7	4.8	3.4	16.8	7.4	6.7	14.6	29.6	11.7	18.5	9.8	13.0		
		49+00	7.1	2.8	2.8	12.7	7.0	9.4	14.7	43.6	18.5	27.7	15.7	22.0		
10	I	49+50	6.9	6.2	5.0	26.2	6.3	7.5	14.3	27.5	11.7	21.9	9.8	16.0		
		50+00	21.4	6.1	2.9	15.6	6.8	7.2	13.7	26.3	10.9	18.0	10.0	13.0		
		50+50	26.7	8.1	4.1	16.1	6.2	8.1	11.6	19.0	9.3	15.1	5.9	14.0		
12	IV	52+00	18.1	5.6	5.2	20.8	8.2	8.6	17.4	16.1	9.9	18.4	5.2	12.0		
		52+50	8.2	11.3	10.1	30.0	8.7	10.9	12.7	8.0	4.8	14.6	0.0	8.0		
13	IV	54+50	27.4	5.2	5.4	22.6	8.4	7.4	11.7	11.9	5.9	12.7	0.0	6.0		
		55+00	31.2	5.7	4.1	19.4	7.8	6.4	10.3	15.1	5.6	12.6	3.5	9.0		
		55+50	29.8	6.5	5.0	20.6	5.2	7.2	9.7	16.0	6.2	12.7	4.6	12.0		
14	I	56+00	28.9	5.2	3.8	24.8	6.6	6.9	9.3	14.4	7.1	15.9	5.9	12.0		
		56+50	3.2	0.2	0.5	48.0	9.0	6.9	9.5	22.8	19.0	40.0	12.7	25.0		
		57+00	17.7	2.8	2.6	35.9	7.1	5.7	9.3	18.9	12.2	21.9	9.1	27.0		
14	I	58+50	37.8	8.6	8.3	19.0	5.5	5.2	8.0	7.5	4.2	8.7	0.0	8.0		
		59+00	30.4	8.9	8.7	24.2	7.2	6.5	8.1	6.1	2.8	9.5	0.0	7.0		
15	IV	59+50	27.4	6.6	5.6	27.2	5.8	6.2	8.1	12.9	7.6	13.2	3.9	16.0		
16	IV	62+50	21.1	8.7	6.0	23.4	7.3	8.7	9.7	15.0	6.9	11.5	0.0	11.5		
17	IV	64+50	31.8	9.2	3.8	20.5	4.5	4.7	10.5	15.0	6.4	9.8	6.6	8.0		
		65+50	29.6	7.6	4.5	25.8	6.8	5.7	9.6	10.4	4.1	11.6	0.0	6.0		
18	IV	66+00	19.6	7.8	4.5	29.6	7.7	7.5	12.0	11.2	5.1	12.9	0.0	6.0		
		66+50	15.8	7.7	6.2	31.0	6.5	7.7	11.9	13.3	5.4	12.8	0.0	8.0		
		67+00	18.6	9.9	5.8	26.4	7.8	7.2	13.5	10.7	5.5	12.2	0.0	3.0		
		67+50	32.6	12.0	6.7	22.4	4.2	5.2	8.2	8.5	4.8	8.7	0.0	8.0		
19	IV	68+00	16.9	7.2	4.2	24.3	9.8	13.2	10.6	13.8	6.5	11.7	0.0	9.0		
		68+50	5.8	1.9	2.9	25.0	15.4	17.1	14.5	17.4	6.3	18.5	0.0	10.0		
		69+00	9.5	3.5	2.6	26.9	13.4	13.4	15.6	15.1	6.7	12.8	0.0	8.0		
		69+50	10.8	3.5	3.0	24.9	9.7	9.5	16.3	22.4	8.0	13.9	4.6	9.0		
20	IV	70+00	8.8	4.2	2.2	31.8	12.9	16.1	11.1	12.7	4.4	11.5	0.0	6.0		
		70+50	21.7	8.0	5.8	29.1	7.3	6.2	10.4	11.5	5.5	12.7	0.0	7.0		
		71+00	21.6	7.4	5.7	27.2	6.8	7.7	11.6	11.9	5.4	13.8	0.0	9.9		
		71+50	12.4	3.5	2.3	18.7	4.6	5.6	22.2	30.5	10.3	14.5	5.5	9.9		
21	V	72+00	17.6	2.1	1.6	17.5	5.4	6.4	25.9	23.4	7.8	13.6	0.0	4.0		
		72+50	18.1	5.8	4.6	26.8	9.2	12.0	13.4	10.0	22.6	11.0	0.0	3.0		
		73+00	9.0	6.3	6.8	31.9	8.6	10.9	16.0	10.3	4.4	13.7	0.0	4.0		
		73+56.5	33.8	8.2	5.0	21.8	6.4	6.1	9.4	9.2	4.0	11.1	1.8	8.0		
22	V	74+00	12.8	6.4	3.9	25.0	10.8	10.4	13.8	16.9	6.7	15.1	0.0	7.0		
		74+50	10.7	4.6	2.8	24.4	13.7	14.6	12.4	16.9	7.7	15.7	5.1	9.0		
		75+00	10.6	5.7	4.0	26.5	11.0	16.1	11.9	14.2	5.0	18.5	0.0	8.0		
		75+50	32.6	8.2	4.8	28.4	5.1	3.9	6.5	10.4	5.4	8.0	0.0	8.0		
23	V	76+00	3.3	1.5	1.3	27.0	19.3	20.8	14.5	12.1	6.4	17.1	0.0	9.0		
		76+50	1.5	1.1	0.9	26.5	20.3	21.8	13.3	14.6	4.6	20.1	0.0	9.0		
		77+00	2.6	1.0	0.7	26.5	16.4	23.4	13.2	16.1	7.6	16.2	0.0	9.0		
24	V	77+50	4.4	0.8	0.8	18.4	13.8	20.3	14.8	26.7	9.8	26.0	10.9	12.0		
		78+00	8.0	8.2	7.2	31.5	11.3	10.8	14.6	8.2	3.1	22.3	0.0	3.0		
		78+50	30.8	10.6	6.2	23.6	5.2	5.9	8.2	9.5	3.9	15.1	0.0	7.0		
		79+00	22.7	6.5	5.4	28.8	7.8	7.1	10.4	11.3	4.7	18.0	0.0	6.0		
		79+50	32.2	9.1	5.5	21.9	5.5	6.5	9.8	9.5	3.2	14.7	0.0	6.0		
25	VI	80+00	4.8	4.2	5.6	32.0	11.6	11.4	19.5	11.0	4.8	21.7	0.0	3.0		
		80+50	15.0	5.4	3.3	25.1	9.9	9.2	16.4	15.7	6.6	20.5	0.0	8.0		
		81+00	4.4	2.1	2.4	20.5	9.9	11.4	23.0	26.1	9.7	22.1	0.0	7.0		
		81+50	22.4	8.2	5.6	25.8	6.6	8.1	9.2	14.0	5.0	15.9	0.0	4.0		
26	VI	82+00	8.1	3.7	2.6	17.5	8.7	10.1	20.5	28.8	9.9	21.0	8.6	9.0		
		82+50	0.0	0.3	0.4	9.7	4.9	8.3	31.0	45.4	15.5	28.4	11.5	16.0		
		83+00	0.0	0.3	0.5	14.7	7.9	11.5	30.9	34.2	12.0	22.7	7.8	12.0		
27	VI	83+50	0.0	2.1	0.3	13.4	6.5	9.7	23.2	14.8	17.2	30.4	16.5	28.0		
		84+00	0.0	0.4	0.8	15.2	7.1	8.4	20.1	18.0	16.7	27.4	12.0	21.0		
		84+50	2.3	1.0	0.9	17.0	8.3	10.4	27.8	32.2	9.6	23.3	12.8	9.5		
		85+00	9.2	4.1	2.8	15.6	6.0	7.0	25.2	30.2	7.8	27.1	11.9	8.5		
29	1	88+00	21.4	7.3	6.2	29.2	7.6	8.9	11.1	8.3	2.5	17.7	0.0	4.0		
		88+50	7.0	6.0	7.1	27.8	8.6	10.2	18.1	15.2	6.7	25.1	0.0	11.0		
29	1	89+00	30.0	5.8	4.8	22.5	6.0	6.6	10.9	13.2	5.1	17.9	7.0	11.0		
		89+50	15.9	5.9	5.7	24.8	7.1	9.0	16.6	14.9	6.3	22.8	7.5	10.5		
		90+00	8.3	8.2	8.5	33.2	8.0	9.1	14.6	10.2	3.5	22.6	0.0	6.5		
		90+50	1.6	0.3	0.7	17.0	8.5	9.2	20.7	42.0	13.7	35.9	12.3	18.0		
		91+00	32.0	6.0	2.6	18.8	5.7	5.2	9.8	19.9	7.3	20.2	7.1	12.0		
29	1	91+50	35.8	4.5	2.7	23.4	5.4	3.8	9.1	15.1	6.5	13.8	4.7	11.0		
		92+00	31.2	10.3	5.9	21.9	4.5	4.6	7.0	14.4	4.7	15.8	0.0	11.0		
		92+50	28.5	4.9	3.3	21.4	6.3	5.1	12.1	18.3	5.8	19.7	6.1	10.4		
29	1	93+00	29.0	5.8	3.5	18.9	5.5	5.4	12.2	19.6	7.4	15.8	8.9	13.0		
		93+50	27.6	4.0	2.5	20.4	5.8	5.4	13.1	21.2	8.3	17.7	9.4	11.0		
		94+00	19.7	2.2	1.4	19.1	6.8	6.8	17.2	26.7	10.7	17.6	7.0	14.0		
29	I	94+50	34.8	3.1	1.8	18.6	6.6	5.5	10.3	19.2	8.0	17.3	7.8	17.0		
		95+00	25.5	8.1	7.7	24.2	7.1	7.4	12.0	6.9	2.6	13.8	0.0	7.0		
		95+50	6.4	4.2	5.9	27.7	7.1	7.6	19.5	21.8	9.2	24.5	6.9	11.0		
		96+00	7.6	4.5	11.1	29.6	8.8	12.5	17.5	8.4	4.0	20.3	0.0	7.5		
		96+50	34.8	6.2	4.2	13.8	3.0	3.5	12.7	21.8	7.7	14.8	5.6	12.0		
		97+00	41.9	4.4	3.9	14.4	3.7	3.9	11.3	16.5	6.6	15.4	5.6	10.0		
29	I	97+50	29.7	6.8	5.5	20.9	3.5	4.3	10.9	18.2	6.3	15.0	5.9	8.0		
		98+00	40.3	4.0	3.7	19.7	3.9	3.6	10.4	13.7	5.8	12.7	3.2	8.0		
		98+50	34.0	6.2	4.8	19.4	5.4	5.2	11.5	13.3	5.7	15.1	5.3	10.0		
		99+00	44.2	9.2	4.8	15.0	3.3	3.6	8.6	11.3	5.5	12.5	3.3	10.0		
29	I	99+50	35.3	6.9	3.4	20.7	3.6	3.8	9.5	16.6	6.7	17.1	5.8	15.0		
		100+00	46.3	11.7	5.2	18.1	2.3	2.6	5.4	8.4	3.8	9.2	0.0	10.0		
		100+50	26.7	3.9	3.5	24.0	5.8	5.5	12.9	17.5	8.8	20.5	7.9	13.0		
		101+00	32.8	4.2	3.9	21.1	4.0	4.6	10.7	18.7	8.3	15.4	4.7	14.0		
29	1	101+50	24.4	1.7	2.1	13.8	4.9	8.2	20.3	24.5	14.9	23.5	9.5	40.0		
		102+00	23.4	2.1	2.2	12.8	5.6	7.4	19.4	27.1	14.5	21.5	9.0	46.0		
		102+50	22.6	2.3	1.7	15.2	6.9	8.3	19.4	23.4	10.6	16.5	6.7	18.0		
		103+00	8.1	2.8	3.8	26.0	7.6	9.4	21.9	20.2	9.4	26.3	9.4	15.0		
29	I	103+50	26.5	2.4	1.6	10.8	4.3	5.3	19.7	29.3	19.2	29.9	14.0	61.0		
		104+00	9.1	0.4	0.0	4.8	1.2	7.9	48.7	27.8	31.4	3.1	16.0	42.0		
		104+50	15.1	0.3	1.8	6.1	4.7	13.4	33.2	25.3	26.0	35.7	14.5	64.0		
		105+00	24.2	1.7	1.2	8.2	4.3	6.8	18.7	35.0	21.0	27.7	15.2	67.0		
29	1	105+50	16.0	3.6	1.8	11.0	4.6	8.0	23.0	31.9	21.8	30.5	17.1	65.0		
		106+00	32.8	5.1	3.7	13.3	4.4	6.1	14.6	19.9	11.8	19.9	13.2	32.0		
		106+50	22.9	2.1	2.1	11.4	4.3	8.6	20.8	27.7	17.8	27.1	14.8	47.0		
		107+00	19.8	4.3	4.9	18.2	8.6	9.9	21.2	12.9	9.4	19.7	4.8	15.0		
		107+50	4.7	3.2	4.0	21.2	9.2	16.7	27.0	13.9	8.3	25.8	7.4	24.0		
		108+00	5.4	6.7	4.2	20.7	7.7	13.5	19.9	21.8	9.2	23.4	9.8	10.0		
30	VII	108+50	5.9	1.3	3.3	10.4	3.4	5.0	22.1	48.5	31.7	39.2	22.1	73.0		
		109+00	13.2	3.9	3.3	8.2	2.9	5.4	20.7	42.3	23.1	31.7	19.1	82.0		
		109+50	22.0	7.5	4.5	19.3	6.7	10.2	10.9	18.7	9.8	15.5	6.0	13.0		
31	VIII	110+00	19.5	8.6	5.7	30.0	7.1	5.9	8.0	15.2	6.6	16.6	4.4	10.0		
		110+50	12.2	3.4	2.9	16.1	10.2	14.8	15.5	24.8	9.8	18.3	7.0	13.0		
		111+00	8.3	3.3	1.6	13.9	17.3	18.4	14.9	22.1	10.5	21.1	7.8	12.0		
		112+00	24.3	12.2	5.6	20.7	6.0	7.5	8.9	14.8	5.9	13.4	0.0	12.0		
32	IX	112+50	11.6	2.0	2.3	13.2	6.6	11.7	18.9	33.5	14.9	21.9	10.2	27.0		
		113+00	9.4	1.1	1.6	5.5	2.9	12.5	24.0	42.9	29.3	36.6	20.8	110.0		
		113+50	24.5	4.1	3.8	8.4	1.7	3.3	19.5	34.7	20.7	27.0	15.6	85.0		
32	IX	114+50	22.7	7.7	3.7	12.5	2.2	4.3	16.5	30.4	15.1	23.9	12.3	23.0		
		115+50	23.1	8.8	5.2	23.0	5.8	9.1	10.7	14.2	8.2	18.1	9.4	14.0		

[1] Comparing these analyses with those of the soils examined by A. C. Rose as reported in the article entitled "Practical Field Tests for Subgrade Soils," PUBLIC ROADS, volume 5, No. 6, August, 1924, the following differences will be noted: The analyses here reported were made with Tyler standard mesh sieves instead of metric sieves. The samples were analyzed just as they were taken from the subgrade without removing the coarse material. Silt, as defined by Mr. Rose, is material finer than 0.05 millimeter and coarser than 0.005 millimeter; as here used the term refers to material passing a 200-mesh sieve which is not held in suspension by water. Clay, according to Mr. Rose's definition, is material finer than 0.005 millimeter. As here used the term refers to material held in suspension by water. The maximum size of particles is approximately 0.010 millimeter, about twice as great as the maximum size of the particles designated as clay under Mr. Rose's definition.



TYPE I

PLAIN CONCRETE SECTIONS

SECTIONS 1, 5, 14 AND 29 HAVE 6 INCH SIDE THICKNESS AND 7-INCH CENTER THICKNESS

SECTION 8 HAS 7-INCH SIDE THICKNESS AND 8-INCH CENTER THICKNESS

SECTION 11 HAS 8-INCH SIDE THICKNESS AND AND 9 INCH CENTER THICKNESS

LEGEND

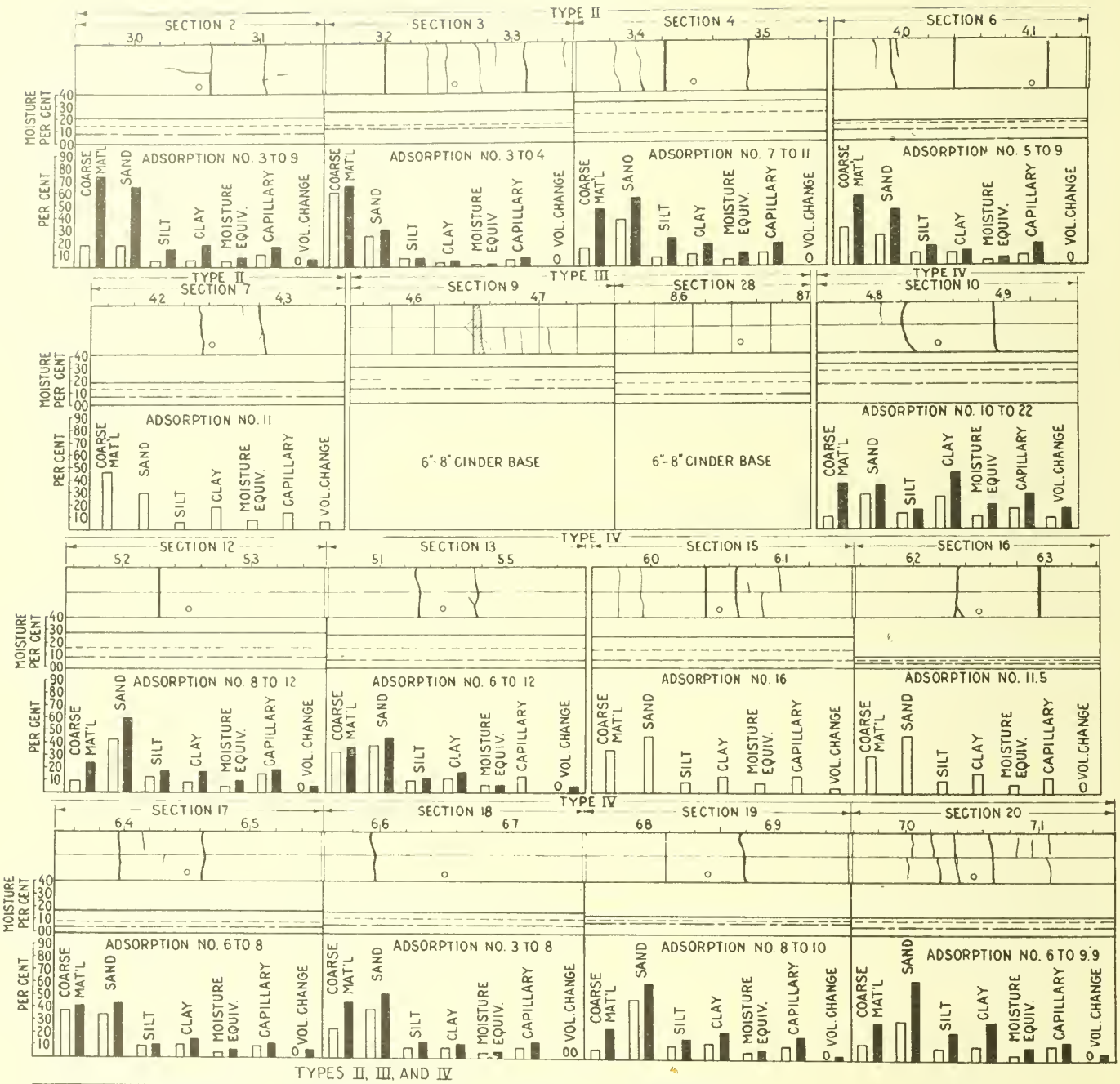
- ○ INDICATES POINTS AT WHICH CORES WERE DRILLED AND MOISTURE DETERMINATIONS MADE.
- —— INDICATES MAXIMUM MOISTURE PERCENTAGE
- ---- INDICATES AVERAGE MOISTURE PERCENTAGE
- - - INDICATES MINIMUM MOISTURE PERCENTAGE
- ~~~ INDICATES CRACKS IN PAVEMENT AS REVEALED BY SURFACE SURVEY.

SURFACE SURVEY : MAY 9, 1924

MOISTURE DETERMINATIONS MADE:

JULY 31, 1922
AUGUST 7, 1922
JANUARY 15, 1923
JANUAPY 23, 1923
JANUARY 30, 1923
FEBRUARY 28, 1923
MAY 19, 1923
MARCH 10, 1924
APRIL 17, 1924

FIG. 2.—Crack records of Type I (plain concrete) sections, mechanical analyses of their subgrades, and subgrade moisture determinations



TYPES II, III, AND IV

TYPE	SECTION	THICKNESS (INCHES)	REINFORCING		JOINTS		CONSTRUCTED BASE
			LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE	
II	2	6					
	3	7					
	4	8					
	6	6	25 LBS. PER 100 SQ FT. MESH				
	7	6	50 LBS. PER 100 SQ FT. MESH				
III	9	4			CENTER	30 FT C TO C	8"-6" CINDER
	28	6			DO	DO	DO
IV	10	6			DO		
	12	7			DO		
	13	8			DO		
	15	6	8 ½ INCH ROUND	3/8 INCH ROUND 12" C TO C	DO		
	16	6	DO	3/8 INCH ROUND 18" C TO C	DO		
	17	6	DO	3/8 INCH ROUND 24" C TO C	DO		
	18	6	4-½ INCH ROUND		DO		
	19	6	4-¾ INCH ROUND		DO		
	20	6	4-1 INCH ROUND		DO		

LEGEND

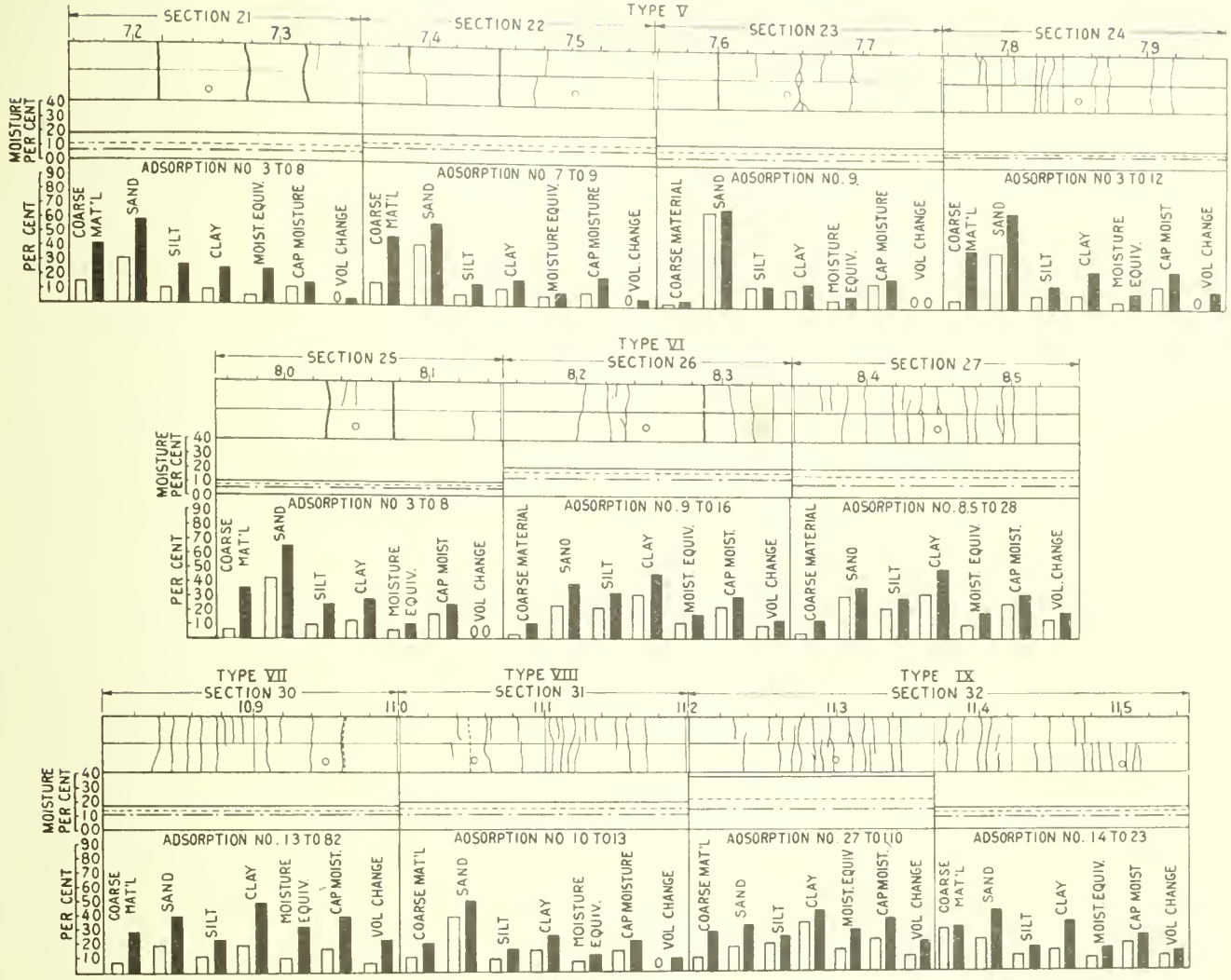
- ○ INDICATES POINTS AT WHICH CORES WERE DRILLED AND MOISTURE DETERMINATIONS MADE
- —— INDICATES MAXIMUM MOISTURE PERCENTAGE
- - - INDICATES AVERAGE MOISTURE PERCENTAGE
- — — INDICATES MINIMUM MOISTURE PERCENTAGE
- —~— INDICATES CRACKS IN PAVEMENT AS REVEALED BY SURFACE SURVEY

SURFACE SURVEY: MAY 9, 1924

MOISTURE DETERMINATIONS MADE:

JULY 31, 1922	FEBRUARY 28, 1923
AUGUST 7, 1922	MAY 19, 1923
JANUARY 15, 1923	MARCH 10, 1924
JANUARY 23, 1923	APRIL 17, 1924
JANUARY 30, 1923	

FIG. 3.—Crack record of sections of Types II, III, and IV, mechanical analyses of their subgrades, and subgrade moisture determinations



TYPES V TO IX INCLUSIVE

TYPE	SECTION	THICKNESS (INCHES)	REINFORCING		JOINTS		CONCRETE RIBS	
			LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE	LONGITUDINAL	TRANSVERSE
V	21	6			CENTER	NONE	2 TYPE A	NONE
	22	8			00	00	00	00
	23	6	8-3/4-INCH ROUND		00	00	00	00
	24	6	00	3/8-INCH ROUND-18 INCH C TO C	00	00	00	00
VI	25	6			00	00	4 TYPE A	00
	26	6	8-3/4-INCH ROUND		00	00	00	00
	27	6	00	3/8-INCH ROUND-18 INCH C TO C.	00	00	00	00
VII	30	6	SHOWN ON PLAN		00	20 FEET C TO C	6 TYPE B	9 FT. 10 IN. C TO C
VIII	31	6	14-3/4-INCH ROUND	1/2-INCH ROUND-18 INCH C TO C	00	NONE	2 TYPE A	NONE
IX	32	6	00	00	00	00	NONE	00

SURFACE SURVEY : MAY 9, 1924
MOISTURE DETERMINATIONS MADE :
JULY 31, 1922
AUGUST 7, 1922
JANUARY 15, 1923
JANUARY 23, 1923
JANUARY 30, 1923
FEBRUARY 28, 1923
MAY 19, 1923
MARCH 10, 1924
APRIL 17, 1924

LEGEND
- ○ INDICATES POINTS AT WHICH CORES WERE DRILLED AND MOISTURE DETERMINATIONS MADE
- —— INDICATES MAXIMUM MOISTURE PERCENTAGE
- - - - - INDICATES AVERAGE MOISTURE PERCENTAGE
- —·— INDICATES MINIMUM MOISTURE PERCENTAGE
- —— INDICATES CRACKS IN PAVEMENT AS REVEALED BY SURFACE SURVEY

FIG. 1.—Crack record of sections of Types V to IX, inclusive, mechanical analyses of their subgrade, and subgrade moisture determinations.

Subgrade materials which have a large percentage of granular material and very little clay not only take up less moisture during the wet season than those which are high in clay but also retain less during the dry season. This is shown in sections 9 and 28 where the subgrade is cinders, and in sections 25, 24, 23, 19, and 16 where a large percentage of clean sand is found.

Whether the subgrade is on a cut or fill has some effect on the amount of moisture it will hold. Subgrades through a cut will attain higher moisture and retain more of it than similar subgrades over a fill. Section 30 on a high fill and 32 in a cut indicate the effect of this difference on the moisture content of a subgrade.

The percentage of capillary moisture as determined by examination of a subgrade sample by the present laboratory test does not represent the maximum amount of moisture that will be held by the particular subgrade. Comparison of results of moisture determinations in Table 3 with capillary values given in Table 2 shows the capillary values to be about 75 per cent of the maximum moisture and about equal to the values obtained in the subgrade during the warm season. This can probably be accounted for by the fact that the laboratory test is made at room temperature and that capillary moisture increases as the temperature is reduced to freezing. It would seem that the laboratory test should be made on the samples when they are close to freezing and not, as is now done, at room temperature.

The lack of stability of subgrades which are composed of materials showing high volume change is very definitely shown by certain sections. In the plain concrete sections those whose subgrade materials show volume change in the laboratory have cracked longitudinally, while those sections whose subgrades show no volume change have not cracked. Section I has cracked badly longitudinally. Although the subgrade shows some volume change, some of the cracks are probably due to one or more springs in the subgrade. Evidence of the existence of such springs is the free water observed along the edges and in the cracks.

Section 29, stations 103–105, has a subgrade similar to the adjoining sections but at the time of construction cement in the proportion of 1 to 28 was mixed to a depth of 6 inches with the soil. This section has no longitudinal cracks. The high moisture readings in the adjoining sections are also obtained in this section but much less moisture is retained in the treated subgrade. The effect of the treatment on this section is quite evident when compared with the two adjoining sections, which have cracked badly longitudinally. This treatment has apparently so modified the character of the subgrade that volume changes so destructive in the adjoining section have not done any damage in this one. It is very likely, however, that a layer of cinders or other granular material such as coarse sand would have provided approximately the same results.

Materials which undergo large changes in volume seem to be those which have a large percentage of clay. The quantity and character of this clay, as indicated by the adsorption number of the material, is also indicative of its action under a change in moisture content. Sections which have a large amount of granular material, as illustrated by sections 3, 4, 5, show no volume change, others show changes as high as the 16.8 per cent in section 27. Other sections which are high in volume change are 10, 11, 15, 26, and 29, stations 89–91, 93–97, 101–108, and sections 30, 31, and 32. Sections which have high volume change have a high adsorption number, while the reverse is true where the volume change is small.

TABLE 3.—*Moisture content of subgrades at intervals since the construction of the Columbia Pike*

Section No.	Station	Moisture content at certain dates									
		July 31, 1922	Aug. 7, 1922	Jan. 15, 1923	Jan. 23, 1923	Jan. 30, 1923	Feb. 28, 1923	May 19, 1923	Mar. 10, 1924	Apr. 17, 1924	May 19, 1924
1	28+50	[1]15.3	[1]13.0	14.0	12.0	[1]18.6	[1]21.3	[1]18.6	[1]17.9	[1]21.5	[1]18.6
2	30+50	7.8	7.0	9.7	15.0	16.4	17.0	17.8	16.2	20.4	21.1
3	32+50	11.9	10.3	10.9	13.3	13.1	12.4	12.8	24.2	15.2	16.8
4	34+50	7.6	6.8	21.0	16.2	26.3	30.1	30.4	28.8	31.9	25.9
5	36+50	15.7	12.3	7.7	10.5	20.9	24.4	20.2	10.9	13.6	21.0
	38+50	7.7	8.1	9.6	9.6	21.9	22.1	20.6	15.2	20.1	19.0
6	41+00	9.1	8.6	13.9	15.4	13.9	14.4	15.3	12.6	8.0	16.0
7	42+50	7.6	6.9	14.8	10.1	12.9	14.9	15.4	17.3	16.9	15.2
8	44+50	9.5	8.9	11.8	10.8	14.5	30.4	24.9	17.0	13.4	12.6
9	46+50	16.7	16.1	22.0	17.6	16.2	16.5	12.2	26.2	15.9	29.7
10	48+50	15.1	18.3	25.0	27.3	27.5	29.3	27.7	30.7	23.4	24.1
11	50+25	29.5	29.9	28.0	29.2	25.2	31.9	30.4	41.2	33.7	33.2
12	52+50		9.8	9.1	10.0	18.6	22.2	26.5	18.1	16.4	19.0
13	54+50	6.4	8.8	18.3	18.6	15.9	21.6	20.1	13.2	26.0	24.8
14	56+50	9.1	13.3	16.6	22.5	20.2	22.7	19.9	22.3	22.8	21.9
	58+50	7.2	9.7	14.5	16.7	17.1	17.1	14.7	11.2	18.3	12.1
15	60+50	8.4	7.3	12.9	13.2	12.8	11.2	11.0	20.6	24.0	19.9
16	62+50	7.0	6.8	7.7	7.6	8.1	7.7	7.2	5.9	8.2	6.8
17	64+50	7.5	5.8	7.7	8.0	9.0	9.8	8.9	7.1	15.9	17.1
18	66+50	7.5	7.7	7.0	15.3	9.4	14.9	13.8	13.0	10.2	7.3
19	68+50	8.6	9.6	10.4	10.3	10.4	12.2	14.0	13.8	12.8	11.8
20	70+50	6.5	14.8	10.0	9.9	11.6	10.0	9.0	10.3	12.3	16.5
21	72+50	8.7	7.2	7.6	8.5	17.6	9.2	11.8	9.2	13.7	17.0
22	75+00	7.8	8.0	8.3	9.0	13.7	13.4	12.3	12.5	17.6	10.4
23	76+50	5.7	5.6	5.9	9.6	10.5	8.9	6.8	7.8	10.1	12.2
24	78+50	9.3	9.3	9.9	10.0	9.0	11.2	15.3	8.8	00.0	11.8
25	80+50	6.0	6.7	7.2	7.9	9.6	8.9	9.9	5.9	8.4	9.9
26	82+50	12.1	12.5	12.4	15.5	13.6	19.7	17.4	15.8	19.0	14.5
27	84+50	8.3	9.2	12.4	16.2	15.0	16.4	12.3	13.2	18.0	13.8
28	86+50	8.2	9.7	20.0	21.4	24.8	25.2	16.5	15.5	16.1	23.3
29	88+50	5.2	6.0	7.1	8.5	8.0	8.4	11.4	13.4	16.6	10.4
29_1	90+50	14.9	13.9	19.3	15.1	15.8	12.6	14.0	14.1	18.5	20.6
29_2	92+50	10.3	11.4	17.9	15.9	18.4	15.7	16.7	20.0	22.2	18.4
29_3	94+50	13.8	15.3	17.1	19.0	19.2	22.5	18.9	19.7	17.9	18.4
29_4	96+50	6.7	7.0	26.1	18.2	30.6	19.9	22.8	17.8	25.2	13.0
29_5	98+50	8.2	9.6	21.0	12.0	19.9	20.3	20.4	13.9	18.2	25.4
29_6	100+50	12.6	12.2	17.2	16.9	19.5	20.4	14.9	18.0	17.2	17.8
29_7	102+50	14.0	13.9	17.4	16.3	10.2	18.4	16.8	20.9	00.0	18.4
29_8	104+50	21.1	13.1	41.0	27.3	34.7	30.2	26.8	38.3	43.4	28.9
29_9	106+50	26.4	25.4	43.0	39.8	20.2	31.0	37⅛6	38.5	26.5	30.6
30	109+50	10.8	12.6	11.4	15.1	13.0	12.0	11.6	11.0	10.1	11.7
31	110+50	13.1	15.8	9.5	13.4	17.8	11.6	19.8	15.6	15.5	12.7
32	113+00	14.7	28.8	36.0	33.7	16.8	18.0	13.9	17.8	14.1	15.1
	115+00	13.1	12.1	14.7	7.9	13.4	12.2	8.5	7.9	00.0	14.5

[1] Free water in subgrade.

LONGITUDINAL JOINTS PREVENT LONGITUDINAL CRACKS

Those sections in which longitudinal joints have been constructed are in all cases free of longitudinal cracks, even when the subgrade material, as in sections 10, 15, 24, 26, and 27, changes in volume. There is this distinction, however, that in the section with a high volume change the longitudinal joints are open, showing that more or less movement has taken place at the edge with the joints acting as a hinge; where there has been no volume change, the appearance of the joint indicates little if any movement about the center axis.

Transverse cracks resulting from contraction occur at intervals which correspond to the distance in which the force of subgrade friction becomes greater than the tensile strength of the concrete. Unevenness of the subgrade such as might be obtained by construction of the pavement on rough macadam, for example, increases the number of transverse cracks by increasing the friction between the subgrade and pavement.

An increase in thickness of the pavement does not materially affect the spacing of the transverse cracks as caused by contraction in the pavement. The additional strength of the thicker section is probably balanced by the additional friction on the subgrade. Types of construction of plain concrete which have the section strengthened by additional thickness or thick-

ened edges, although they do not materially affect the spacing of transverse contraction cracks, do serve to keep down the number of additional cracks caused by heavy traffic.

TRANSVERSE CRACKING NOT PREVENTED BY LONGITUDINAL STEEL

One of the purposes of this experiment was to investigate the behavior of sections with different kinds and quantities of steel reinforcing. The results obtained to date indicate emphatically that longitudinal steel, above a certain amount and installed as it generally is now, does not prevent transverse cracking but does, on the other hand, greatly increase the number of such cracks.

Figures 2, 3, and 4 show that there is a great difference in the appearance and location of the transverse cracks. In sections 27, 30, 31, and 32, where the percentage of longitudinal reinforcing is large, the surface shows transverse cracks every few feet. Sections 30, 31, and 32 having a higher percentage of steel than No. 27 show a greater number of cracks. The effect of the amount of longitudinal reinforcing on the pavement is further indicated in sections 15, 16, 17, 18, 19, 20, 23, 24, and 26. In these sections the amount of reinforcing is varied; comparing them, we notice that 23, 24, and 26, each of which have eight ¾-inch rods, are not cracked so badly as sections 30, 31, and 32 which have a larger number of rods. Section 20, with four 1-inch rods, is cracked slightly less than the sections which have eight ¾-inch rods. Sections 15, 16, 17, 18, and 19, with reinforcing varying in amount from four ¾-inch rods to four ½-inch, are free from the fine cracks. These sections have open cracks similar to those found in a plain concrete section but spaced at greater distances (60 or 70 feet). Sections 6 and 7, which are reinforced with 25 and 50 pounds of mesh per 100 square feet, have only the open cracks spaced at a greater distance than would ordinarily be found in plain concrete.

From the condition of the reinforced sections it would seem that some doubt can be properly expressed as to the value of longitudinal reinforcing as it is now used in the pavement. The longitudinally reinforced sections in this experiment show conclusively:

1. That longitudinal steel up to a certain amount does increase the spacing of the transverse cracks and that these cracks are open on contraction.
2. That longitudinal steel above this amount will produce fine transverse cracks spaced in some cases only a few feet apart.

DEDUCTIONS WITH REGARD TO TRANSVERSE CRACKING

The action of these sections and the results of observations on other concrete pavements seem to support the following deductions:

In plain concrete the movement of the pavement in contraction is resisted by the friction between the slab and the subgrade. This force carried to the pavement is very high, much higher than would be expected as the movement is very slow. On the average subgrade it will crack the pavement in tension at intervals from 40 to 60 feet. In cases where the subgrade is rough and offers high resistance to the movement of the slab, transverse cracks will develop at shorter intervals. An extreme case of this was observed when a concrete pavement was laid on a rough natural rock base, the pavement being badly damaged by numerous transverse cracks. It has been observed that the width of these cracks varies directly as the distance between them. In cases where the transverse cracks are close together and very fine, they become wider as time goes on from the grinding and crushing of the edges of the crack. This is particularly apt to occur in cases where the subgrade happens to be unstable.

In the case of pavement reinforced longitudinally, the resistance to contraction developed by the subgrade is similar to that in the plain concrete. Along with this force there is the added force developed by the bond of the steel. Acting against this pair of forces is the force resulting from contraction limited by the tensile strength of the concrete. Taking the case of a slab with continuous reinforcing, we would have as we move from one end toward the other increasing resistance to movement of the slab over the subgrade and increased force from the restrained contraction of the pavement. These forces increase as the distance from the end of the slab or crack is increased until the tensile strength of the concrete is exceeded, producing a transverse crack in the slab. The stress in the concrete is thereby relieved, but the steel remains in tension and continues to oppose the contraction of the concrete beyond the first break. With respect to this section of the concrete the stress in the steel is an initial force to which the subgrade frictional forces are added, with the result that in a comparatively short distance the strength of the concrete is again exceeded and another crack is formed. Repetition of this process produces a series of closely spaced cracks, until the accumulating tensile force ruptures the steel.

If the steel is not continuous and the joints which separate it are close enough together, it is possible that there may be no cracking of the slab. This condition would be expected in pavements which are separated by joints into sections the length of which is less than twice the distance required for the formation of a first crack. There are no such sections in the Columbia Pike, but sections 24, 26, 27, 30, 31, and 32 supply evidence of the validity of the above theory. Their first cracks occur about 40 feet from the joint, while the other intermediate cracks are spaced from 10 to 15 feet apart.

The effect of accumulating tension in the steel is illustrated by the behavior of sections 6, 7, 16, 17, 18, 19, and 20, in which large open cracks have developed. Opening these cracks at points where the steel was located, it was found that in sections 23, 24, 26, and 27, in each of which the longitudinal steel consists of eight ¾-inch deformed round bars, no change in the diameter of the rods had occurred as far as could be determined. In section 20, which is reinforced with four 1-inch rods, slight reductions in diameter were found. In section 19, with four ¾-inch rods, the diameter was reduced to five-eighths inch, and there was evidence of a nearly complete failure in the steel. In sections 16 and 17, with eight ½-inch rods, the diameter was reduced to approximately three-eighths inch, while in section 18, with four ½-inch rods, there was a complete break with a reduction in the diameter to about one-fourth inch. In sections 6 and 7, with 25 and 50 pounds of mesh per 100 square feet area, complete breaks were found at the cracks.

THE COST OF GRADING WITH FRESNOES

SOME SUGGESTIONS AS TO COMMON LOSSES AND HOW TO AVOID THEM

By J. L. HARRISON, Highway Engineer, United States Bureau of Public Roads

FROM the management standpoint the outstanding problem on a construction job is how to obtain high production. The time required to perform each necessary operation and the degree to which unnecessary operations are eliminated are the determining elements. Large losses in time are more apt to be the result of a small loss on each performance of an operation necessarily repeated many times than of any outstanding error in the performance of an important operation. No foreman of a grading job would think of letting one of his teams stand idle on the job all day, but he may produce much the same effect by disregarding as small a detail as that of allowing all his teams, after dumping, to swing through an arc with a 30-foot radius when a 15-foot radius would do just as well. The loss of time in this case, being small in each instance, is apt to escape notice. High production can be secured only when such losses are reduced to a minimum, and in order to make them as small as possible their causes should be studied in detail.

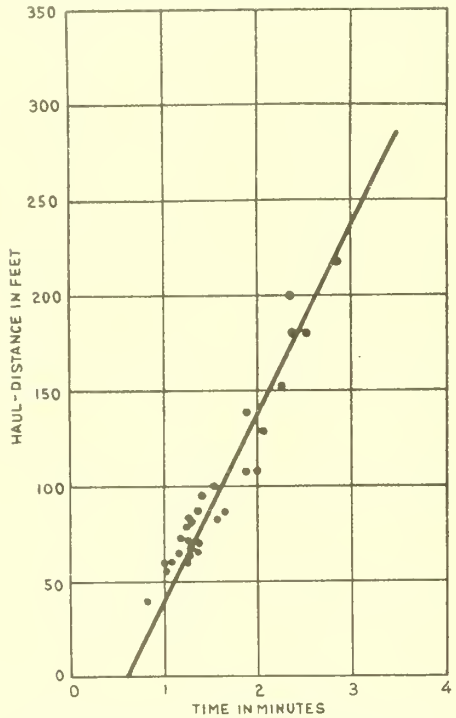


FIG. 1.—A well-managed fresno job. Note consistency of performance and low time loss on turning, loading, etc.

Approaching from this angle the problem of moving earth by fresno, the typical operations are: (*a*) Loading, (*b*) hauling to the dump, (*c*) dumping and spreading, (*d*) returning for the next load, and (*e*) turning to load again. The order of these operations may be varied. For instance, the load may be taken before the turn for the return trip is made; but the sequence is of less importance than the nature of the operations themselves because, whatever the sequence, each takes time and is, therefore, a source of possible loss.

The studies which have been made by the Bureau of Public Roads indicate that the losses suffered by contractors doing fresno work fall within the two categories of managerial losses and bidding losses. The managerial losses are those which can be corrected by carefully systemizing the work. On a fresno job they include those due to improper adjustment of equipment to the work, short loading, time lost in loading, time lost in turning, careless dumping, and a number of miscellaneous losses not directly related to these. The haul itself offers little opportunity for management, as mules set a fairly steady pace which the studies indicate to be little affected by external conditions.

Bidding losses which are, perhaps, as important as the managerial losses, are those which are suffered because of incorrect bidding. They arise principally from a failure to gauge correctly the influence of haul.

SOME CAUSES OF SHORT LOADING

Referring first to the managerial losses, one of the most significant is short loading. It takes just as long to move half a load as it does to move a whole load. Therefore, if a proper output is to be obtained, the loading must be kept up to standard. The bureau's studies indicate that a standard 4-foot fresno loads about a third of a cubic yard per trip. On a well-managed job the fresnoes will be loaded to their standard capacity every trip. A part of the load may be dropped during a long haul, but, from the standpoint of the practical contractor, the effect of this is relatively small, because he can easily allow for it by leaving the grade a little low if much hauling is to be done over it. In any event he generally finds it necessary to go back over his work and correct it for shrinkage, etc., when the final stakes are set; and the material that falls off the fresnoes usually serves to reduce by so much the amount that must be handled in the clean-up. Short loading, however, is a positive loss the causes of which should be sought and eliminated wherever possible. It may be due to the indifference or inefficiency of the drivers. When this is the case the drivers should be trained to load properly or be replaced.

Poor plowing is often an element in short loading. It also affects the time spent in loading. If the soil is properly plowed, a full load can be secured in sand unless it is very dry, in loam, and in clay except where it is unusually heavy. But plowing is relatively expensive and is hard on the stock—particularly plowing in that sort of heavy ground where it is most needed if full loads are to be taken without undue loss of time. In such cases the problem presented to the foreman is a choice between the tangible reduction in output which will result if a fresno is laid off and the mules are hitched to the plow, and the intangible losses which result from slower and lighter loading when there is inadequate plowing.

Light loading and loss of time in loading sometimes result also from failure to remove the roots of trees. If the contract price for earth excavation is 20 cents per cubic yard and labor is 25 cents an hour, an increase in output of 12½ yards will pay for a 10-hour day's work in removing roots. With an ordinary small outfit of six to eight fresnoes working on a 100-foot drag, the loss of as little as 0.05 minute per load or of 2 per

cent in the amount loaded is sufficient to justify the employment of an extra laborer to remove roots or obstructions of any other character which interfere with proper loading. The losses due to obstructions are often much greater than this.

On short hauls every effort should be made to secure heavy loading. Whatever additional yardage may be secured in this way is a clear gain. It is not advisable, however, to attempt heavy loading on long hauls, because of the greater amount of work such hauls impose on the stock. In a haul of 100 feet, the round-trip time averages 2.2 minutes. Twenty-seven trips are made in an hour. Only one-half minute of the trip time is spent in hauling the load to the dump, which means that the team is under full load 13½ minutes per hour. On a 300-foot haul the time required per trip is 4.2 minutes, and there are 14 loads per hour. In this case the team is under full load 1½ minutes per trip or 21 minutes per hour. The fact that on the longer haul the stock performs 50 per cent more heavy work during the day than on the short haul offers a reasonable explanation of the fact that—even without extra-heavy loading—stock generally shows a distinct loss of weight if held on long-haul work for a protracted period.

All "pestering" of stock should be rigidly avoided and teamsters who engage in it should be replaced. Some teamsters abuse the stock under the pretense that they are ambitious for more speed. Jobs where this is permitted almost always show an increase in the time taken per trip.

CONSISTENCY OF PERFORMANCE AN IMPORTANT ELEMENT OF GOOD MANAGEMENT

One of the more conspicuous facts brought out by the studies is that the difference between good management and average or poor management is largely a matter of the consistency of the performance. Good management sets a high standard and consistently follows it. Under average or poor management the results secured will at times compare favorably with what would be expected under the best of management, while at other times the results will fall so low as to be classed as poor. The lack of consistency can be traced to a tendency on the part of many foremen to allow the physical conditions surrounding the job to govern performance. If these favor a high output, a high output is secured, but if they tend to create losses no adequate effort is made to avoid their effect. So, if the lay of the work suggests that a long swing at the dump be made, the long swing is not prevented by managerial effort, though the loss of but a tenth of a minute on each load may easily mean the loss of a hundred yards on the day's output. The tendency of foremen to allow physical conditions to establish the details of performance which they should themselves govern may in some cases be due to a lack of energy but it is more apt to be a matter of lack of training. However, the point it is desired to emphasize is that a contractor should view inconsistency in output on a fresno job as indicating weak field supervision and take immediate steps either to develop or to displace a foreman whose record of performance is of that kind.

The difference between good and average management as measured by the results obtained may be illustrated by the following example: A normal fresno job will show an average haul of perhaps 150 feet. On such a job the average output under average management would be about 73 cubic yards per fresno per day of 10 hours. Under good management it would be about 95 cubic yards per fresno per day. With a 10-fresno outfit the difference in output would be 220 cubic yards per day, which, at 25 cents per cubic yard, is $55 per day. In other words, on such a job as this a saving of one-tenth of a minute per load means a saving from $7.50 to $10 a day for the contractor.

So far no significant differences have been found in the rate at which such materials as sand, light gravel, loam, clay, or even heavy clay are moved once the load is taken. There is little variation in the pace of the mules either in moving the material or returning from the dump. There appears to be a little slowing

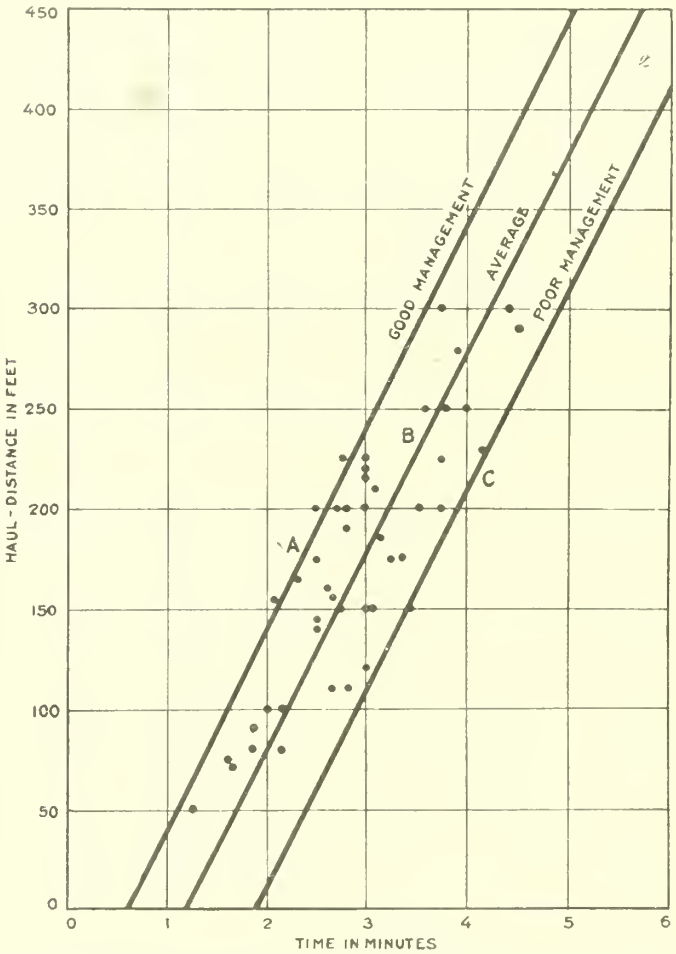


FIG. 2.—Diagram comparing job represented in Figure 1 with other jobs typical of average and poor management. The points shown illustrate lack of consistency on a job where the control was weak

down in loose, dry sand and a little speeding up on material which packs readily and gives a particularly smooth, hard footing, but in the Mississippi Valley, where the bureau's studies have been made, such materials are exceptional. On the other hand, very stiff clay loads with difficulty even when well plowed, and there is a distinct tendency for the men to tire when handling it. Graphs, similar to Figure 1, based on morning and afternoon observations on the same well managed job, show the effect of the increased time required for loading, unloading, and turning in the afternoon by increases in the X intercept. The uniform pitch of the rate lines brings out the fact that on heavy work the stock continues to move at its regular pace after the men have begun to show fatigue. Under conditions

involving heavy work in loading it is well to put on an extra man to load the fresnoes in order to relieve the teamsters.

WET WEATHER LOSSES LARGELY AVOIDABLE

Among the managerial losses those incident to wet weather should be mentioned. It is customary to stop work whenever there is rain and to delay reopening until the ground has dried out more or less. During such delays the stock has, of course, to be fed. Many contractors also feed their men without charge, and it occasionally happens that labor conditions are such that the men also must be paid. It should, therefore, be noted that fresno work can be performed without great difficulty even in mud so deep that the mules sink above their fetlocks. Nor is the work as much delayed by such conditions as might be supposed. The mules move a little more slowly and there is a tendency to lose time in loading and dumping, but outputs can be readily secured which more than cover operating costs. The

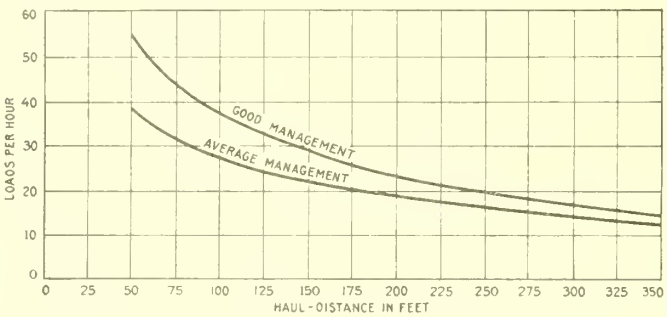


FIG. 3.—Diagram showing number of loads per hour under good management and average management.

dead loss of the corral and the mess is thus avoided even though no profit is made. After a heavy shower work should be started at once. The surface of the ground will be muddy but a single cut will remove the mud, and even if the wet earth must be cleared from the path over which the teams move the actual cost of this operation is small as compared with the cost of delay. As soon as the wet surface is removed, a fresno job can be made to run as smoothly as in dry weather.

Another loss that can be easily avoided is the maintenance of extra stock. It is not uncommon to find that an outfit is maintaining 3 to 5 extra animals in the corral as against 25 to 30 on the job. On the other hand, one job (not a fresno job) now under study maintains 44 horses on the job with a reserve of one in the corral. With proper attention to the selection of good drivers and with proper attention to the stock there is small need for more than one extra animal unless the outfit is unusually large. At present, feeding a mule costs from 60 to 70 cents a day or about $20 a month. Any animal which has a tendency to "go lame" or injure itself on the work is, therefore, too expensive to keep.

Figure 2 compares the results of good, average, and poor management. Graph A shows the results on a job where the foreman used conspicuous care in avoiding time losses of all kinds. Short diameter turns were insisted on and the equipment was kept in motion. Graph B shows the average results on a considerable number of fresno jobs. The intercept on the X axis, a constant which covers loading, unloading, and turning, with incidental losses of time, is 0.6 minute per load on the well-managed job as compared with 1.2 minutes under average conditions and 1.9 minutes with poor management. There is no significant difference in the rate at which the teams move to and from the dump. The effect of the differences which these graphs show as between the "loading, dumping, and turning time" under good and average management is of considerable importance as the following table, based on a 100-foot haul, shows.

	Good management	Average management
Time per load, 100-foot haul, minutes	1.6	2.2
Loads per fresno per 10-hour day	375	272
Loads per day for eight-fresno outfit	3,000	2,176
Per cent of average performance	138	100
Quantity of excavation per day, cubic yards	1,000	725
Teams required to produce 1,000 cubic yards per day	8	11

In Figure 3 the value of close attention to the elimination of lost time in loading, unloading, and turning, with incidental losses, is shown in another way in two graphs which give the number of full loads per hour per fresno which can be expected under good management and under average management. Using these graphs any foreman engaged on fresno work can determine the efficiency of his outfit by the following simple process:

1. Count the number of loads dumped during an hour.
2. Divide the number of loads dumped by the number of fresnoes at work to obtain the average number of loads dumped per fresno.
3. Find the average distance over which the loads dumped during the count were moved.[1]
4. Compare the average number of loads per fresno, obtained as indicated in 1 and 2, with the number shown by the graphs (or Table 1) for the distance determined as indicated in 3, to ascertain whether the management is above or below average.

Table 1 gives the same information contained in Figure 3 for intervals of 25 feet.

TABLE 1.—*Fresno output*

[Standard 4-foot fresnoes]

Haul distance (feet)	Loads per fresno per hour		Haul distance (feet)	Loads per fresno per hour	
	Good management	Average management		Good management	Average management
50	54½	38	200	23	19
75	44	31	250	19½	16
100	37½	27	300	16½	14
125	32½	24½	350	14½	13
150	28½	22	400	13	11½
175	25½	20			

BIDDING LOSSES

Aside from the losses incurred through faulty management, contractors often suffer serious losses because

[1] In practice, when moving earth from a cut to a fill the area over which loads are being secured and the area over which dumping is being done are often quite sharply defined. For the purpose of testing the efficiency of operation it is accurate enough to measure from the center of the area over which loading was done during the count to the center of the dumping area, a procedure which has been found by experience to give results agreeing within a few feet of the average distance obtained by taking the distance on each load and averaging these distances.

of careless bidding on fresno work. It is customary to call for bids for highway excavation on the cubic-yard basis, and quite commonly contractors are influenced in determining the amount of their bids by the unanalyzed cost of other jobs. Presumably this practice is based on the presumption that moving one cubic yard will cost about as much as moving another, or, in any event, that the average work required in moving a cubic yard of earth on one project will be approximately equal to the average work required on another project. The fact is that the unanalyzed cost of work done on a past project is a dangerous criterion of the cost of prospective work because it takes no account of the haul which, in practice, varies from mile to mile on an individual project, and varies considerably as between different projects. The force of this statement will be seen by a little study of the typical fresno loop (see fig. 4) or by a study of the time-distance graph (fig. 2), and is clearly developed in the right-hand column of the detailed time schedule and estimate on page 15.

If the drag or distance moved (see fig. 4) is long, a large amount of time is consumed in dragging one load and returning for another. If the drag distance is zero, then theoretically a load can be picked up and dumped at any point on the approximate circle resulting from a combination of the two turning swings with small loss of time over what is required in driving around a circle 30 to 50 feet in diameter. This is about what actually takes place when material is moved from the ditches to build up the subgrade. With allowances for the extra effort required in climbing from the ditch to the top of the embankment, as well as for loading and dumping, operations that result in changing the path of travel from a circle to a short loop, the time per trip may, with good management, be kept as low as 1 minute per load, and 1.2 minutes per load is a fairly common performance. Work handled in this way is known as "in-and-out" work.

As against this, a 300-foot drag will require with good management about 3.6 minutes and with average management 4.2 minutes. In other words, a drag approaching the maximum limit of the fresno's range of efficiency takes nearly four times as much time as "in-and-out" work. This illustration will serve to make it clear that the price bid on one job is not a valid criterion of the price which ought to be bid on the next job, unless it is known that the average haul is the same in both cases or that the average time required for a load is the same. Estimates should, therefore, be made up from the plans for the job itself. In no case should they be made from the unanalyzed bids on other jobs.

A METHOD OF SCHEDULING HIGHWAY GRADING

Another matter which ought to receive more attention in connection with highway work is the time scheduling of construction operations. A first-class building contractor will schedule a million-dollar building so closely that he will know its date of completion almost to a day. In doing this he lays down in advance the force his superintendent will be authorized to employ, and, on this basis, calculates the time that will be required in order to do each part of the work. Once this is done, to insure himself that a job is being built at the assumed cost, he has to examine only three things:

1. His materials purchases to see that in quantity and in price these agree with his estimate. As these are usually protected in advance of making his bid, he should find no trouble here.

2. The wages that are being paid. Prevailing wages are, of course, known before the estimate is made, and the superintendent should be appropriately limited in this field.

3. The amount of work done.

If the first two are as planned and the job, in time, is up to schedule, the contractor knows without consulting detailed cost data that the job is being profitably conducted and what the profit is.

The graphs presented in this article offer a ready means of preparing such a schedule for highway grading with standard 4-foot fresnoes and at the same time estimating the cost of fresno work in such a manner that the contractor may at all times feel confident of the results which his field forces are securing. The methods are as follows:

1. To determine the time required on any job—

(1) Multiply the total yardage (excavation plus borrow)[2] by the time required for "loading, dumping, and turning," which is, for average performance, 1.2 minutes.[3]

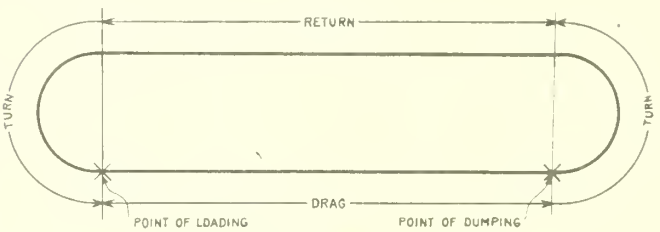


FIG. 4.—Typical loop made by fresno outfit

(2) Add to this the product obtained by multiplying the net yardage of each cut, or borrow pit, by the average distance in stations (100 feet) that it must be hauled.[3] If no haul distance is shown for borrow multiply borrow yardages by 0.5 (one-half station).[4]

(3) Divide the sum obtained above by 200 to obtain the number of days' work to be done, if the working day is 10 hours (180 for a 9-hour day and 160 for an 8-hour day). The method here described is adopted for the sake of simplicity. To obtain the correct time of the job in minutes the extensions under (1) and (2) should also be multiplied by 3, in which case the sum should be divided by 600.

(4) Divide this by the number of fresnoes it is planned to work and the result will be the number of days' work which will be required of the outfit under normal conditions.

[2] Borrow as used here covers material secured outside the limits of the standard cross section—generally though not always by widening ditches.

[3] This product is one-third of the time required for the operation, since it is obtained by multiplying yardage by the time required to handle one-third of a cubic yard. See explanation under (3).

[4] Theoretically side borrow involves only transverse haul. In practice, because ditch lines and the road section generally must be kept reasonably uniform, whereas the ground will vary a good deal, more or less longitudinal haul is required.

(5) To this add one-fifth to cover Sundays, holidays, and wet weather, in order to obtain the working period. For example:

		Good management	Average management
Loading, dumping, and turning time, minutes		0.6	1.2
Excavation, cubic yards	70,000		
Borrow, cubic yards	30,000		
Total earth to be moved, cubic yards [1]	100,000	60,000	120,000
Cut 20,000 cubic yards moved 2.47 stations		49,400	49,400
Cut 10,000 cubic yards moved 3.16 stations		31,600	31,600
Borrow 30,000 cubic yards moved 1.46 stations		43,800	43,800
Cut 10,000 cubic yards moved .74 stations		7,400	7,400
		192,200	252,200
Divide sum above by 200 to give days' work to be done		961	1,261
Divide days' work to be done by number of fresnoes in outfit to give days' work for outfit (in this case 10)		96	126
Add one-fifth for Sundays, holidays, and bad weather		19	25
Working period, days		115	151

[1] By reason of side-hill work and excavation from ditches (in-and-out work requiring only loading, dumping, and turning time) this quantity will generally be greater than the sum of the cuts and borrow pits listed below.

2. To convert this estimate of the time needed to accomplish a specific amount of work into a bidding estimate the next step is to determine the cost per day of operating an outfit of the size used in making the time estimate in this case 10 fresnoes. The costs and wages given below are for illustration only. The contractor must use those actually prevailing.

A typical outfit would be—	Mules	
10 fresnoes, 3 mules each	30	
2 plows, 4 mules each	8	
Extra stock	2	
	40	[5] $35
Add one-fifth for corral cost on Sundays, holidays, and bad weather		7
Superintendent (one-fifth added)		10
Foreman (one-fifth added)		5
Labor:		
12 teamsters, at $3		36
2 plow holders		7
1 laborer (water boy, etc.)		2
Total cost per day		102

On long hauls one plow can be converted into a fresno outfit which gives a small working margin. The above is illustrative only. The contractor must develop his estimate of the cost of a day's work on the basis of the outfit he has and wages prevailing when the work is to be done.

3. Continuing with the example, the time estimate shows the days' work for the outfit to be 96 days under good management and under average management, 126. Multiplying these by the total cost per day, $102, as indicated above, gives a direct expenditure of $9,792 under good management and $12,852 under average management. To this direct expenditure it is essential that four items be added, as follows:

4. The proportional part of the cost of wintering the stock which is properly allocable to the job. These are: Good management, $1,200; average management $1,600.[7]

5. Depreciation on stock and equipment: A 40-mule outfit generally represents an investment of $8,000 to $10,000. Few contractors can get an average of more than five years' work out of a mule. He must then be sold and replaced. His sale value will seldom be above $50. The annual depreciation per animal is, therefore, in common practice, from $30 to $40. That part of this depreciation properly chargeable to this job should be added, as should the probable repairs and depreciation on equipment. The latter is not a heavy item on fresno jobs; so, for the purposes of this example, the depreciation to be added may be assumed to be: Good management, $1,400; average management, $1,600.[6]

6. Moving costs: It costs a good deal to get from one job to the next. If the job is long enough to constitute a season's work the cost of getting back to winter quarters should be added. The costs are estimated as follows: Good management, $250; average management, $500.

7. Profit, estimated as follows: Good management, $3,000; average management, $3,000.

Recapitulating, the various items of cost are as follows:

	Good management	Average management
Direct expenditure for job	$9,792	$12,852
Wintering stock	1,200	1,600
Depreciation of stock and equipment	1,400	1,600
Moving costs	250	500
Profit	3,000	3,000
Total value of the job	15,642	19,552

Dividing these total costs by the total yardage moved, which is 100,000 cubic yards (excavation plus borrow), the prices which should be bid under the two conditions are as follows:

Good management ..... 15½ cents per cubic yard.
Average management ..... 19½ cents per cubic yard.

This estimate is based on the presumption that nothing but plowing is done by the plow teams. With small outfits—about 6 fresnoes—this is generally true, the plow team being allowed to stand idle a good deal of the time whenever the haul is long and the amount of earth which must be moved by the fresnoes is correspondingly small. With larger outfits, as the haul increases plow teams can be cut off and put at hauling. However, as outfits operating more than 10 fresnoes in a unit are seldom encountered on highway grading work the better practice in estimating is to consider the plow teams as used only on plowing, because there is little assurance that any large part of the time of the plow teams can be diverted to hauling.

Heavy clay, gumbo, and other extra-heavy materials, if dry, require more than the ordinary plow equipment, and, even after good plowing, load more slowly. This difference can be covered in the estimate of the cost of a day's work (1) by allowing for the extra stock and men required to do the extra-heavy plowing and (2) by allowing for extra men to do the loading, thereby relieving the teamsters and enabling them to keep the job up to schedule in other respects.

It is worthy of note that if a job is scheduled and estimated according to the plan outlined above, no extra allowance is needed for overhaul, as the overhaul is included in the bid price.

Such items as clearing and grubbing, maintenance of grade during the construction period, the cost of the clean-up, etc., are given no treatment here because the specifications of the various States differ so materially

[5] This item should cover all corral costs such as labor in caring for animals, feed, veterinary services, etc., but not depreciation on stock or equipment.

[6] For purposes of illustration it has been assumed that under average management this would be a full season's work, but that under good management it would not.

in these particulars that it has been considered advisable merely to refer to them and to state that such items, when included as part of the grading should be set up in the estimate as separate items just as depreciation, getting onto the work, etc., are set up.

A SAMPLE GRADING SCHEDULE

As a more complete illustration of these methods, there follows a schedule and an estimate based on an actual project. This schedule and the estimate, for purposes of illustration, are based on the presumption that fresnoes will be used throughout the job and that the management is equal to the average. In practice both wheelers and fresnoes are likely to be used on a job of this sort, the former being used wherever the haul much exceeds 350 feet. There is a little haul of this kind on this project but as it in no way affects the principles which are being discussed, this fact has been disregarded in preparing this schedule.

Detailed time schedule and estimate for a typical highway grading job using a 10-fresno outfit

Mile	Work	Quantity	Time units	Days work one fresno	Days work for outfit	Progress schedule	Unit price per cu. yd.
				Days	*Days*		*Cents*
1	Excavation	4,363.8				Start May 1.	
	Borrow	2,028.4					
	Total excavation	6,392.2×1.2	7,670.6				
	Borrow	2,028.4×0.5	1,014.2				
	Cut	145.4×2.7	392.6				
	Do	121.6×1.6	194.6				
	Do	167.6×1.75	293.3				
	Do	1,480.1×3.0	4,440.3			Finish May 8.	
		Divide by 200	)14,005.6(	70.0	7		17.1
2	Excavation	8,534.5					
	Borrow	2,549.1					
	Total excavation	11,083.6×1.2	13,300.3				
	Borrow	2,549.1×0.5	1,274.5				
	Cut	297.5×3.0	892.5				
	Do	302.5×2.0	605.0				
	Do	778.5×2.16	1,681.6				
	Do	1,193.2×2.10	2,505.7				
	Do	93.6×1.75	163.8				
	Do	12.6×3.0	37.8				
	Do	1,029.3×2.5	2,573.3				
	Do	1,735.4×2.17	3,765.8				
	Do	1,404.7×2.0	2,809.4			Finish May 26.	
		Divide by 200	)29,609.7(	148.0	15		21.2
3	Excavation	8,358.8					
	Borrow	2,224.9					
	Total excavation	10,583.7×1.2	12,700.4				
	Borrow	2,224.9×0.5	1,112.5				
	Cut	536.5×1.5	804.8				
	Do	902.5×1.75	1,579.4				
	Do	1,530.0×2.98	4,559.4				
	Do	69.4×1.95	135.3				
	Do	1,329.9×2.74	3,643.9				
	Do	2,504.8×2.6	6,512.5			Finish noon, June 14.	
		Divide by 200	)31,048.2(	155.2	15½		23.0
4	Excavation	8,096.6					
	Borrow	4,239.9					
	Total excavation	12,336.5×1.2	14,803.8				
	Borrow	4,239.9×0.5	2,119.9				
	Cut	1,084.2×3.0	3,252.6				
	Do	968.5×2.5	2,421.3				
	Do	348.5×2.85	993.2				
	Do	1,661.3×2.86	4,751.3				
	Do	285.9×1.25	357.4				
	Do	615.0×2.00	1,230.0				
	Do	2,860.4×3.00	8,581.2			Finish noon, July 9.	
		Divide by 200	)38,510.7(	192.5	19		24.1

Detailed time schedule and estimate for a typical highway grading job using a 10-fresno outfit—Continued

Mile	Work	Quantity	Time units	Days work one fresno	Days work for outfit	Progress schedule	Unit price per cu. yd.
				Days	*Days*		*Cents*
5	Excavation	2,911.9					
	Borrow	5,761.9					
	Total excavation	8,673.8×1.2	10,408.6				
	Borrow	5,761.9×0.5	2,880.9				
	Cut	648.0×3.0	1,944.0				
	Do	218.2×3.6	785.5				
	Do	302.8×2.98	902.3				
		Divide by 200	)16,921.3(	84.6	8½	Finish July 18.	15.3
6	Excavation	12,773.0					
	Borrow	961.8					
	Total excavation	13,734.8×1.2	16,481.8				
	Borrow	961.8×0.5	480.9				
	Cut	146.6×1.5	219.9				
	Do	646.8×3.0	1,940.4				
	Do	1,705.4×3.0	5,116.2				
	Do	1,581.7×1.5	2,372.6				
	Do	896.1×1.5	1,344.2				
	Do	1,237.0×1.5	1,855.5				
	Do	2,658.7×3.1	8,242.0			Allow 1 day for bad weather; finish noon	
	Do	1,736.9×3.0	5,210.7				
	Do	430.5×3.3	1,420.7				
		Divide by 200	)44,684.9(	223.4	22½	Aug. 15.	25.7
7	Excavation	7,340.1					
	Borrow	8,273.5					
	Total excavation	15,613.6×1.2	18,736.3				
	Borrow	8,273.5×0.5	4,136.7				
	Cut	229.8×1.75	402.2				
	Do	286.3×3.01	861.8				
	Do	1,493.5×2.66	3,972.7				
	Do	2,801.6×2.80	7,844.5				
	Do	187.7×1.7	319.1				
		Divide by 200	)36,273.3(	181.4	18	Finish noon Sept. 6.	18.0
8	Excavation	6,339.5					
	Borrow	9,237.7					
	Total excavation	15,577.2×1.2	18,692.6				
	Borrow	9,237.7×0.5	4,618.8				
	Cut	280.0×1.5	420.0				
	Do	705.8×2.9	2,046.8				
	Do	1,074.0×2.58	2,770.9				
	Do	1,510.5×3.0	4,531.5			Allow 1 day for bad weather; Finish	
	Do	308.3×2.6	801.6				
	Do	502.2×2.22	1,114.9				
		Divide by 200	)34,997.1(	175.0	17½	Sept. 27.	17.6
	Totals: Excavation	58,718.2	246,050.8	1230.1	123		20.5
	Borrow	35,277.2					

Total earth to be moved, 93,995.4.

123 days work @ $102	[1] $12,550
Winter coast of corral	1,600
Depreciation on stock and equipment	1,600
Moving to and from job	500
Profit	3,000
Bid price for job	19,250
Bid price per day ($19,250 divided by 123 days work)	156.50
Bid price per cubic yard ($19,250 divided by 93,995.4 cubic yard)	.205

[1] See estimate of value of day's work, page 14. Extensions to nearest $10 only.

The bid price per cubic yard which is developed by the above method is an average price in which all haul, including overhaul, is merged and in which no differentiation is made between borrow and excavation. Most contractors make little or no difference between borrow and excavation in presenting their bids and overhaul is generally underbid where the free-haul limit is within the fresno haul limit. Engineers are

(Continued on page 23.)

SAND-CLAY AND SEMI-GRAVEL ROADS STUDIED

PROGRESS REPORT OF COOPERATIVE RESEARCH ON SELECTED ROADS IN GEORGIA

IN JULY, 1922, a cooperative study of 29 Federal-aid projects in Georgia was begun by the State Highway Department of Georgia and the United States Bureau of Public Roads, the University of Georgia also participating. The work has been under the immediate supervision of C. M. Strahan, director of research, State Highway Department of Georgia, who in a progress report gives the scope of the work and the progress thus far made.

The 29 projects selected for study comprise a total of 184 miles, distributed generally over the State, and are representative of the variations in topography, climate, and soil conditions in the State. The research has for its economic aim the collection of data as to first cost, annual maintenance, residual worth, traffic density served, and period of efficient service with which to make economic comparison with other types of road, to fix the limits of cost and traffic density to which these types are best adapted, and to guide judgment as to replacement with the same or other materials. It has for its purpose in the physical field the determination of the causes of strength and weakness, the development of tests and standards, and the hope of experimental discovery whereby improved and more dependable road service can be secured from abundant and inexpensive sources of road-building material.

At the beginning of the investigation complete records were available on the cost of each project which had been constructed under engineering supervision in accordance with specifications approved for Federal-aid construction in Georgia. Since completion the projects had been maintained by the counties but were then in process of being put entirely under State control, thus permitting control of maintenance methods and allowing complete cost records to be kept.

The research was planned to cover the following program:

1. Inspection trips each spring and fall. On the spring inspection a record is made by means of notes and photographs of any weaknesses developed during the winter and the effect of current maintenance is observed. On the fall inspection samples of road surfacing are secured at selected points on each project for laboratory analysis and study and at the same time road conditions are noted.

2. The taking of systematic traffic counts (separating passenger cars, trucks, and horse-drawn vehicles). The plan has been to secure a 12-month daylight count on each project during eight days of each month, using two days in each week (Sunday and Monday of the first week, Tuesday and Wednesday of the second week, and so on).

3. The collection and study of maintenance costs on each project in the light of traffic counts.

4. The study in the laboratory of the several soil ingredients—i. e., gravel, clay, silt, and sand—with a view to determining more scientifically the physical, chemical, and mass action of such mixtures; to differentiate the binding values of various clays, if possible; and to investigate the interlocking bond strength of the coarser and fine aggregates involved.

It is planned to continue this line of research until the projects have reached the stage of reconstruction or substantial replacement, at which time an estimate of their residual or salvage value can be made to complete the economic showing desired.

The investigation has now been under way for two years and a considerable mass of data has been collected, but it is felt that it is too early to attempt to draw conclusions or to present a detailed analysis of the figures. Table 2, however, serves to give an idea of the character of information being collected. The figures under mechanical analysis of surfacing material represent the average composition of the surface on each project and are based on a total of 184 samples analyzed in duplicate.

For purposes of comparison, the following classification of sand-clay and top-soil mixtures by the Bureau of Public Roads is given.

TABLE 1.—*Characteristics of typical sand-clay and top-soil mixtures*

	Class A (hard)	Class B (medium)	Class C (soft)
	Per cent	*Per cent*	*Per cent*
Sand	65–80	60–70	55–80
Silt	0–15	10–20	10–20
Clay	9–18	15–25	10–25

CHANGES NOTED IN THE COMPOSITION OF ROAD SURFACES

A table similar to that given for the year 1922 has been prepared for the year 1923, and it is interesting to note the changes which have taken place in a year's time as indicated by samples taken at identical locations. The roads are maintained by road machines and drags which dress the shoulders as well as the surface, and the effect of this must be taken into consideration as well as that of wind and rain. Changes in coarse material were only such as might be expected from variation in sampling. The reduction in coarse sand (above No. 60 sieve), anticipated on account of the grinding and wear of traffic, was found to average 4 per cent.

An average loss of 2 per cent in total sand and a gain of 2 per cent in clay are both contrary to expectations. There is no change in the average amount of silt. The increase in clay occurs in all roads except three red-pebble, gravelly roads, in which losses in clay from 1.2 to 2.5 per cent are shown. Another year's wear may change the clay indications, but, assuming that this increase may be confirmed next year, the following comment is pertinent: With red-clay shoulders thinly covered or unprotected by soil, increase of clay would naturally follow from machining the surface. But the roads with sandy shoulders likewise show increase of clay, accompanied usually by the gradual formation of a loose sand covering on the wheelway. The assumption has been that the chief losses from these surfaces would be in silt and clay, the gathering of loose sand being taken as proof. If dry-weather loss from wind and dust alone be considered, this is probably true. But it is conceivable that traffic friction most easily dislodges the sand grains from embedment in the clay and silt; that any loosened clay and silt may tend to rebind aided by dews and light rain; that the washing rains act chiefly on the

loose sand which they move to the side ditches. Hence the net result of both wind and water losses may be to remove a larger weight of sand than of clay. This is the more probable, since the total clay and silt is only 30 per cent while the total sand is from 60 to 70 per cent of the weight of the surfacing. Future samples from the loose material and from the upper 3 inches of the road surface will be taken to throw light on this seeming anomaly of increase in clay content.

A record has been made of the loss in thickness of the road surfaces. Here, too, such losses are the combined result of traffic wear and maintenance operations. The record shows no case where the original thickness has been increased. A maximum loss of 2 inches on one low-grade soil is shown for the year. Those projects with considerable amounts of gravel show the least loss of thickness. The average for all sample points was 0.6 inch. No consistent relation of thickness lost to traffic counts has appeared during the first year's wear. The roads with heavy traffic generally show a larger loss than those with light traffic, where the materials are reasonably comparable.

The figures for 1922 on annual cost per mile for maintenance of roadway and shoulders are based in part on the judgment of the division engineers, having been compiled during the first year of the State's control of maintenance while its organization and equipment were in the formative stage. For succeeding years these figures will rest on a direct account kept for each project. It will be readily seen how such cost data will be cumulatively useful in guiding budget estimates, in measuring the cost of traffic service, and in fixing the proper economic ratio of service to traffic density.

They bear also on problems of partial betterment to prolong the life of these roads under rapidly increasing traffic. Judging from the present record and from observed field conditions, it is tentatively thought that the better grades of road soils may be expected to show satisfactory service for a traffic density of 300 to 400 vehicles per day during a life of from five to seven years with an upkeep figure approaching $250 per mile.

The research studies are already leading to some conclusions which are thought to be well worth considering in designing these types of roads. It is believed that the maximum allowable amount of clay of 18 per cent in class A surfaces (Table 1) may be increased somewhat where more than 10 per cent of material above the 10-mesh sieve is present. Field observations show that the amount of clay desirable is not a fixed percentage of the total amount of fine material or binder since it is affected by the amount of coarse material with which the binder is associated. The extreme maximum of clay seems to be about 25 per cent of the mixture including gravel with a preferable amount of 20 per cent. Reduced to a percentage of the total amount of fine material, this would give a limit of 20 per cent clay where gravel constituted 10 per cent of the surfacing and 25 per cent clay for 30 per cent or more gravel. It has been observed that corrugations and potholes develop most commonly in those surfaces in which the clay percentage is low.

COARSE MATERIAL AND SMOOTH GRADING IMPORTANT

It is uniformly found that the greatest density of road surfaces of this type is in the upper 3 or 4 inches, with decreasing density and increasing moisture as the subgrade is approached. It is very desirable to know the variations of these factors in the road soil surfaces. The real sources of road efficiency in these materials vests in the mass action by which concentrated pres-

TABLE 2.—*Data collected on 29 projects in 1922*

Project No.	County	Length	Type of surface	Thickness of surfacing	Mechanical analysis of surfacing material [1]					Daily traffic		Annual cost per mile for maintenance
					Gravel in total sample	Sand retained on No. 60 sieve	Analysis of material passing No. 10 sieve			Passenger vehicles	Trucks	
							Sand	Silt	Clay			
		Miles		*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*			
1b	Henry	7.0	Top soil	7.5	11	50	76	11	13	800	110	$306
4	Walton	10.8	do	8.2	12	49	71	11	18	90	6	103
5	Bacon	17.6	Sand-clay	5.1	7	64	77	6	17	170	12	182
6a	Hall	5.7	Top soil	5.1	21	45	68	15	17	140	50	203
18	Dooly	10.4	Sand-clay	3.5	3	48	73	10	17	60	25	217
19	Bleckley	9.1	do	5.8	7	43	72	9	19	90	25	----------
22	Milton	10.0	Top soil	6.0	18	36	63	19	18	130	170	156
41	Douglas	6.6	do	3.8	8	52	72	15	13	230	65	88
49	Mitchell	13.4	Pebble	7.4	20	37	73	14	13	140	18	157
59	Jackson	4.3	Top soil	6.8	16	52	69	12	19	280	44	162
60	Stephens	11.3	do	5.4	9	46	74	11	15	230	22	100
68	Walton	9.1	do	9.7	7	53	73	14	13	140	16	114
76	Wheeler	9.9	Sand-clay	8.8	9	54	75	7	18	120	18	131
77	Charlton	19.2	Clay-gravel	8.0	35	60	72	6	22	190	13	188
124	Washington	5.9	Top soil	6.8	6	42	74	8	18	130	14	149
131	Jackson	7.1	do	6.4	12	54	72	10	18	280	44	166
134	Coweta	19.9	do	8.5	5	55	75	13	12	110	13	121
144	Macon	3.8	Sand-clay	6.2	0	39	76	12	12	210	22	207
145	Montgomery	12.4	Pebble	7.4	16	44	68	9	23	140	15	98
146	Tift	6.0	do	1.0	12	45	71	14	15	250	30	200
151	Floyd	7.3	Chert	------	50	34	50	20	30	120	20	61
178	Murray	6.6	do	8.1	38	25	40	25	35	160	20	50
179	Greene	2.7	Semigravel	10.9	23	50	65	21	14	260	26	122
189	Hart	3.8	do	9.9	9	56	75	11	14	310	22	201
196	Early	9.0	Sand-clay	7.8	6	52	74	8	18	120	15	149
197	Bulloch	4.8	Pebble	7.7	14	45	77	9	14	140	10	215
199	Quitman	5.0	Gravel	5.8	26	46	72	11	17	200	28	160
205	Effingham	4.2	Top soil	------	4	40	77	8	15	210	50	350
S-10-14	Richmond	8.4	Clay-gravel	6.3	36	68	75	4	21	300	46	128

[1] The No. 10 sieve is taken as the dividing line between course material and fine material. In analysis of fine material the sample is taken as 100 per cent after the removal of the coarse material.

sures and impacts are distributed and resisted, and in the ability of certain soil mixtures to maintain this mass action in the presence of notable percentages of water or their ability to so limit the ingress of water that the mass strength is not seriously diminished. The power of the road surface to resist water absorption and to avoid a too serious loss of mass stability under impact is the reason why certain soil mixtures are reliable and stable while others are hopelessly worthless.

Convinced by close observation that well-selected class C soils will pack into slabs of ample strength and possess in themselves sufficient resistanec to undue water saturation, the Georgia Highway Department has recommended on several projects where class A materials were not available the following type of construction: An 8-inch bed of available class B or class C local soil to be covered with 2 inches of imported fine gravel, screenings, chats, or like hard material. The latter is applied before the soil bed is well packed. In the outcome the harder surface is securely bonded to the soil bed, consolidation of the road proceeds rapidly and evenly, and a firm slab of excellent durability is formed. The hard surface layer, though thin, protects the soil bed from direct abrasion and from dust and washing losses, which are the chief defects of these lighter grades of road soils. The mass action of the whole slab is strong and satisfactory.

The studies and observations are leading to a greater confidence in the influence on road soil and gravel types of the small-sized hard material lying between three-eighths and one-tenth inch in diameter. Particular attention is called to project 179, Greene County, where a class A binder is reinforced by 23 per cent of hard quartz particles all of which lie between a No. 4 and a No. 10 sieve. This road, which is easily mistaken for a paved road, has borne for three years without perceptible wear a traffic density of 281 vehicles per day, and the maintenance cost has been chiefly for building and upkeep of the shoulders.

Similar evidence is accumulating on some of the clay-gravel roads where material above the No. 4 sieve is less than 20 per cent, the deficiency being made up by the hard material just above the No. 10 sieve.

There seems to be an increase of density and a more uniform mass action where soil and gravel mixtures grade smoothly from coarse to fine. Within certain limits, the absolute size of the coarse aggregate seems less important than the uniformity of the grading. In this climate there appears to be less tendency to corrugation and pitting on gravel roads where a liberal amount of binder unites a graded gravel which starts at or below a 1-inch maximum size.

Close observations are being made on this important point. The local gravel deposits of Georgia are characterized by smaller sizes than the glacial gravels of the North and West. They are more commonly associated with clay binders which grade as class A material. They have shown excellent durability, smoothness, and low cost of upkeep.

EFFORTS MADE TO DEVISE TESTS FOR QUALITY

One of the major purposes of the research is to perfect methods of test for the control of these types of material as used in road construction. Attention has been directed to a review of present standards and laboratory methods and to devising new tests and apparatus in promising directions in addition to the field experiments carried on in connection with the current construction and maintenance work of the Georgia Highway Department. Much attention has been directed toward the study of the behavior of the clay ingredients. Experiments on clay have been made by cementing thin brass plates with clay films. The plates, 1 inch wide, are lapped to give 1 square inch of cemented surface and the break is made by a pull parallel to the surface. It was hoped to measure thus the adhesion of the clay to the plates, or the shear in the clay film, according to the observed fracture. Quantitatively this series of tests has been disappointing. Qualitatively it has been significant of the influence of water on clay behavior. The greatest adhesion was shown by wet films of stiff mud consistency. On drying in air, or completely by artificial heat, adhesion to the plates was rapidly lessened and completely destroyed when dry. The plates were clamped together strongly while drying, but to no avail. The test suggests that the plasticity and adhesion of clay are in large part the result of colloidal films established when water is present.

Wet soil briquettes 1 square inch in area were molded in a divided mold and broken by a tension pull. Using increasing percentages of water, the breaking weights increased to a maximum and then diminished to zero. Considerable differences are noted in the water capacity of different soils in that part of the curve where the tenacity is greatest; also in the absolute amount of the breaking weights.

Of the soils tested, those with low clay content reached a maximum and stayed high with 22 per cent moisture, while those with a larger clay percentage yielded under less pull with about 15 per cent water. The test is quickly made and simple and has value in selecting soils which preserve their stability with high percentages of water.

The test is applicable to dry briquettes, but here the tension figure steadily increases with the percentage of clay and thus draws no line for samples carrying excess clay.

A series of tests, consisting of the breaking of 1 by 1 inch molded bars of soil 5 inches long, has also been made. The soils were mixed to normal consistency—i. e., plastic enough to prevent packing when placed in the molds. The bars were dried in air or on a water bath. The 5-inch bars were broken on a 4-inch span by a transverse load applied at the third points. Most of the bars ruptured at the center of the span or at the loaded points. Each material was tested in duplicate bars. The short broken pieces, four for each material, were further broken in shear by a center knife-edge load on a 1-inch span. Fifty-four soils were tested. The range of breaks for the 4-inch spans was from 2,700 to 15,000 grams. About 60 per cent of these showed a satisfactory agreement of the duplicate breaks (within 10 per cent of the average).

Those which broke in the center of the 1-inch span yielded in every case by diagonal shear toward the point of support. The range of breaks was from 6,000 to 30,000 grams. The four tests on each material gave stisfactory agreement in 50 per cent of the soils tested. The extremely low breaks were associated with the poorer roads, but the medium and high breaks followed no significant relation to the road history of the samples.

A disk shear or punching test was devised and tried as follows: Disks of the soil 2 inches in diameter and

1 inch thick were molded from stiff mud under 3,000 pounds total compression. When dried, the disks were placed in a punching cylinder to fit and fractured by a 1-inch diameter steel plunger acting in line with a 1-inch hole in the bottom of the cylinder. Three disks were made for each material and 31 soils were tested. The breaking loads ranged from 425 pounds minimum to 3,340 pounds maximum. The average agreement of the three disks for each material was fairly good. It was felt, however, that the ratio between the areas of plunger and disk is too large for best results. It is planned to repeat the test using 4-inch disks and a three-quarter inch plunger. A test of this kind is very promising, but the labor of making the disks is considerable.

A METHOD FOR THE DYE-ADSORPTION TEST

The dye-adsorption test for clays, as originally devised by the Bureau of Public Roads, has been applied in the Georgia work. Both crystal violet and methylene blue have been used as the adsorption color media. The standard filter tubes, as used by the bureau, and a colorimeter method have been used. The colorimeter adsorption method was devised in order to use directly a liquid sample from the clay washings. Comparative tests on clay which had been recovered by evaporation to dryness with the same clay tested while still moist showed much higher adsorption in the latter case. Hence it was felt that if the clay washings could be used directly much time in recovering the clay would be saved and the adsorption figures would more closely represent the nature of the original clay. The method proceeds as follows: The clay washings are caught in a large jar and brought to known volume (say 2 liters) with distilled water. From the freshly and vigorously shaken liquid quickly take 25 cubic centimeter samples by a standard pipette. Evaporate one sample to dryness to determine the amount of clay in each sample. Place the other samples in flat-bottom glass flasks and add an excess of standard methylene blue solution (1:1000), recording the amount added. Bring the flask to vigorous boiling and continue for three minutes. Transfer and rinse the flask liquid into a cylindrical graduate and dilute with distilled water to fixed volume, 50 or 100 cubic centimeters. Allow this to settle until perfectly clear and take an accurate pipette sample for use in the colorimeter tube and compare by the usual colorimeter procedure. Having thus determined the residual amount of methylene blue not adsorbed, this amount subtracted from the original addition will give the amount of methylene blue adsorbed.

A batch of 12 samples is easily carried on by this method and duplicate determinations are very close. The adsorption test is very delicate, dealing with minute actual weights of coloring matter and probably influenced by the presence of small amounts of ionizing salts. To be significant in distinguishing clays, the test conditions must be very carefully standardized. It is, however, one of the most promising tests yet proposed for differentiating substances like the clays.

Following the Bureau of Public Roads methods for separation of the total clay into coarse clay and suspension clay by means of the centrifuge, some work has been done on the Georgia road soils. As compared with other records, these clays seem to be quite low in suspension clay. The tests are being continued and some of them repeated. An experiment is in progress to show whether the suspension clay is a fixed quantity in a given sample or whether the coarser particles of clay can be made to pass into the colloidal suspension state.

Another test has been recently planned for field use as the result of preliminary laboratory tests on large slabs of road soil. It may be called a penetration or a bearing-power test. A simple apparatus was devised by which a steel rod one-tenth inch in diameter could be loaded to a ¼-inch penetration when resting on a road fragment about 6 by 6 inches in size. Unit pressures from 4,000 to 7,000 pounds per square inch were recorded on the dry specimens, and 1,600 pounds was held by a road soil with 15 per cent moisture as tested in the field.

A field apparatus similar to an automobile jack but operated smoothly by screw power is being made, with a penetration needle (three sizes), a pressure indicator and a cut-off to regulate the penetration distance. Using the rear-axle weight of his car, the observer with this apparatus will be able to take a large number of bearing-power readings on the surface of the road and at successive depths as desired by digging off the top material. He will also take samples to be put into air-tight bottles for laboratory determination of moisture corresponding to the bearing-power tests. This proposal is similar to the more elaborate subgrade apparatus in use by other investigators for bearing-power tests.

HIGHWAY RESEARCH BOARD TO STUDY CONCRETE REINFORCEMENT

The Highway Research Board of the National Research Council announces the beginning of an investigation of the economic value of reinforcement in concrete roads. Director Charles M. Upham reports that the various State highway departments will cooperate with the board in conducting the investigation. Inspections of pavements will be made in the States of New Jersey, Ohio, New York, Pennsylvania, Delaware, Wisconsin, Iowa, Illinois, and California, and in Wayne County, Mich., and Milwaukee County, Wis.

An effort will be made to determine from a survey of existing roads the influence of steel reinforcement on the resistance of the slab to traffic, subgrade, and climatic conditions; the conditions under which steel reinforcement is especially beneficial to a concrete slab; the effect of slab design on the efficiency of reinforcement; and finally the relative cost of plain and reinforced concrete roads, considering the initial investment, and the annual maintenance and renewal charges.

The procedure will consist of a personal examination of a sufficient number of existing road surfaces to cover different slabs, traffic, and climatic conditions. It is proposed to supplement the examination by photographs, sketches, soil determinations, and other available data. In each case attention will be given to a study of the subgrade to determine its general characteristics and properties as well as the existing drainage conditions.

It is expected that a progress report will be ready for the annual meeting of the Advisory Board on Highway Research to be held at the National Research Council Building, Washington, D. C., December 4 and 5, 1924.

CAUSES OF NONUNIFORMITY OF CONCRETE

A SYMPOSIUM

IN MAY, 1924, the Bureau of Public Roads forwarded to each of its district offices and to all State highway departments a tabulation showing the results of compression tests on cores drilled from several concrete pavements in each of four widely separated States. The test results indicated a lack of uniformity in strength of the concrete taken from the several projects, and the bureau requested those to whom the tabulation was sent to reply with suggestions as to ways and means of improving the uniformity of concrete.

Replies were received from 27 State highway departments and practically all of the district offices. The replies have been analyzed and are now presented in tabular form for the information of those who participated in the discussion and for highway engineers in general. It was hoped by this means to bring out corrective measures of immediate practical application. While the opinions expressed are interesting in their variety, it must be conceded that the majority of the remedies suggested are either quite obvious or lacking in the means of application. The divergence of the opinions is, at least, significant of the complexity of the problem of uniform concrete production.

The subject as a whole is one of prime importance, and it is thought that publishing the information collected will suggest new lines of thought for some and perhaps stimulate investigation and research. All of this should ultimately be conducive to better concrete and better pavements.

Abstract of suggestions of State highway departments and district offices with regard to nonuniformity of concrete as demonstrated by the compressive strength of cores drilled from pavements

Causes of nonuniformity	Suggested by—	
	States	District offices
1. Cement:		
(a) Variation in cement from one plant	2	1
(b) Variation between different brands	3	2
(c) Storage of cement		1
2. Water:		
(a) Variation in quality	2	
(b) Acidity or alkalinity	2	
3. Aggregates:		
(a) Variation in grading	7	6
(b) Variation in cleanliness	2	2
(c) Variation in quality	2	2
(d) Segregation of coarse aggregate in stock piles	2	1
(e) Bulking of sand due to moisture	5	1
4. Construction processes:		
(a) Variation in consistency of concrete	21	6
(b) Inaccuracy in measuring aggregates	12	5
(c) Segregation of concrete in mixer and in pavement	5	
(d) Variation in time of mixing	9	2
(e) Overstanding	1	1
(f) Arbitrary proportions	2	1
(g) Too much tamping	1	
(h) Insufficient tamping	5	
(i) Use of boom-and-bucket mixer	1	
(j) Variable cement factor	1	
(k) Equipment in poor condition	1	
(l) Variation in manipulation	4	1
(m) Variation in pavement thickness	1	1
Curing:		
(a) Variation in later curing due to wet and dry subgrades	1	1
(b) One section shaded, another exposed to sun	1	1
(c) Nonuniform curing	6	3
(d) Variation in nature of material used for cover	1	2
(e) Variable absorption of subgrade	2	2
(f) Inadequate curing	5	2
(g) Absorption by dry subgrade	1	
(h) Temperature changes during curing period		1
(i) Temperature changes during construction period	3	
(j) Rise or fall of temperature during settling period	1	
(k) Effect of freezing	2	

Remedies for nonuniformity	Suggested by—	
	States	District offices
1. Cement:		
(a) Beyond reasonable control	2	
(b) Use only one brand on a project		1
2. Water:		
(b) Test with litmus paper	1	
3. Aggregates:		
(a) Educate material plant personnel	1	
Beyond reasonable control	1	1
Rigid inspection		2
Control by tests	2	
(b) Beyond reasonable control	1	2
(c) Beyond reasonable control	1	1
Control by tests	1	
(d) Stock pile in layers	1	1
Combine separate sizes at proportioning plant	2	1
(e) Weigh aggregates and correct for moisture	1	1
4. Construction processes:		
(a) Slump test	2	
Beyond reasonable control	1	1
Weigh aggregates		1
Accurately measure and proportion water	4	1
Accurately control water for each batch	3	
Add hydrated lime to cement	1	
Use moderately wet mix and then squeeze out excess water.	1	
Closer observation by inspector and foreman	1	
Use mixer which will discharge concrete of proper consistency.		1
Correct leaky valves on mixer		1
(b) Proportion by weight	3	3
Measure accurately and uniformly by means of independent measuring devices.	1	
Use accurate measuring devices	1	
Use batchers	1	
(c) Improve control of consistency	1	
Avoid central mixing plants	1	
Decrease maximum size of stone	1	
Change design of mixers	2	1
(d) Rigid inspection		1
Require 1½-minute mixing period	1	
Require 2-minute mixing period	1	
Use batch meter	2	
Require uniformly definite time of mixing	1	
(e) Proportion aggregates by weight		1
(f) Use scientific proportions	1	
(g) Tamp slightly with not too dry a mix	1	
(h) Use mechanical tamper	3	
Tamp uniformly	1	
(j) Careful inspection	1	
(l) Avoid shoveling and handling after dumping	2	
Use boom-and-bucket mixer	1	
(m) Accurate subgrade and rigid inspection		1
5. Curing:		
(a) Beyond reasonable control	1	1
(b) Beyond reasonable control	1	1
(e) Use tar paper on subgrade		1
(f) Pay more attention to proper curing	1	
Keep pavement wet for at least 14 days	1	
Have contractor cover this item in his bid	1	
(g) Wet the subgrade	1	
(i) Protect work adequately during early fall	1	
(k) Lay no pavement at 40° F. when temperature is falling	1	

Abstract of suggestions of State highway departments and district offices with regard to nonuniformity of concrete as demonstrated by the compressive strength of cores drilled from pavements—Continued

Causes of nonuniformity	Suggested by: States	Suggested by: District offices	Remedies for nonuniformity	Suggested by: States	Suggested by: District offices
6. Engineering supervision:			6. Engineering supervision:		
(a) Poor control and inspection	8	3	(a) Require carefully supervised system of inspection and testing	1	
			Need high-class, intelligent inspection and constant engineering control	1	
			More rigid specifications and enforcement of same through efficient inspection	1	
			Rigid inspection		1
			Adequate inspection		1
			Intelligent inspection		1
			More efficient and competent inspection	1	
			Rigid inspection and control of materials, also accurate and complete inspection record	1	
7. Miscellaneous:					
(a) The main feature yet to be perfected is the accurate measurement of the mixing water				1	
(b) Study of design and specifications required to get uniform working conditions				1	
(c) Extend the laboratory work by relating the test record as nearly as practicable to construction conditions					1
(d) Study means of controlling variable moisture contents of subgrade due to weather and soil conditions				1	
(e) Investigate means of allowing for bulking of sand due to moisture				1	
(f) Surface finish of pavement considered to be a factor of supreme importance				1	
(g) Adequate and competent engineering supervision, and accurate proportioning of materials are the whole solution				1	
(h) Uniform concrete is being obtained as a result of uniform consistency and accurate measurement of materials on all contracts				1	

Abstract of comments on the compression test of cores and the use of the test results as an indication of the quality of concrete in pavements

Comments on compression test	Suggested by: States	Suggested by: District offices	Remedies for inaccuracies of compression test	Suggested by: States	Suggested by: District offices
1. Ends of specimens not true planes perpendicular to vertical axis.	3	1	1. Grind ends of caps	1	
2. Size of specimen not proportional to maximum size of coarse aggregate.	1		2. Cast some sections of pavement 12 inches thick for drilling cores.	1	
3. Variation in speed of test	3	1	3. Use greater care to get uniformity		1
4. Age of specimens not given proper consideration	2	3			
5. Variable diameter of the core		1			
6. Variation in height of cores	2				
7. Injury to cores by drill	3		7. Use molded cylinders for early ages	1	
8. Personal equation of test operators	1				
9. Strain in specimen due to temperature stresses	1		9. Do not drill close to a fracture	1	

Miscellaneous comment (From three States and two district offices):
1. Does the compression test on cores indicate the quality of concrete in the pavement?
2. The core test is unreliable as a criterion of the strength of concrete.
3. The nature of the specimen and the method of test make the core test unreliable.
4. More reliance should be placed on the tensile-strength test than on the compression test.
5. Use compression test only in conjunction with test on beams.
6. Use average of three cores from one locality for the equivalent of one core result.
7. Investigation required to determine reliability of core test.
8. Use beam tests instead of test on cores.

The first and larger part of the tabulation is devoted to the opinions of those who accept the results of the compression test on drilled cores as a measure of the strength of the concrete in the pavement. The numerous recommendations for rigid inspection and close field control stand out prominently among the remedies suggested. This factor, undoubtedly of vital importance, is mentioned by some States in which inspection and control are notoriously weak, which suggests the question as to why, if the weakness is recognized, the necessary steps are not taken to correct the faulty conditions. By comparison of paving work in various localities it is quite apparent that the additional cost of intelligent inspection and a well-planned control of materials and construction is a profitable investment, which is reflected in the quality of the pavements. Without adequate control no scheme for producing better pavements can be properly administered or enforced.

It is of special interest to note that one State claims to obtain uniform concrete as a result of uniform consistency and accurate measurement of materials. This claim is supported by the results of core tests on a number of pavements.

The second part of the tabulation deals with the validity of the compression test on cores. Certain factors are pointed out which may influence the test results to a considerable extent, and it is well that attention be directed toward them. Definite objection is also expressed to the policy of using the results of core tests as an indication of the strength of the concrete. It still remains to be demonstrated that the beam test is a better or more reliable criterion than the compression test on cores. If it is eventually accepted that the beam test is superior, it will then be necessary to develop some convenient and practical method for obtaining beam specimens from the pavement. It is not contended that pavements fail under static load by direct compression, but on the basis of present information it seems reasonable to use the results of tests on cores in a comparative manner for any one pavement as a measure of the strength of the concrete.

The methods employed in constructing the average concrete pavement are in themselves the most convincing evidence that there must be a considerable nonuniformity of concrete. The variation in grading of the coarse aggregate, the lack of accuracy in measuring aggregates, the variable consistency, the shoveling of mortar from fresh batches to supply deficiencies in preceding batches, and inadequate or nonuniform curing are some of the main factors which can be verified by mere inspection and which are apt to contribute to variation in the strength of the concrete. How to correct or improve on these conditions is the problem.

ROAD MATERIAL TESTS AND INSPECTION NEWS

SIEVE LIMITS FOR THE MECHANICAL ANALYSIS OF SUBGRADE SOILS

The subgrade laboratory of the Bureau of Public Roads has been endeavoring for some time to standardize the tests used in determining the physical properties of subgrade soils. One of the essential tests is the mechanical analysis of the soil, and in the consideration which has been given to this test the principal question has been that of the choice of sieves.

The United States Bureau of Soils in all its analyses has employed metric sieves, which separate the soil into constituents the size limits of which are expressed in millimeters. The sieves generally used in highway practice for the analysis of sands, topsoils, and sand-clay mixtures have sizes of opening which are defined by the number of meshes to the inch, and the sizes of opening thus defined differ materially from those employed by the Bureau of Soils.

Notwithstanding the fact that much of the work of previous soil investigators has been based on the metric system, it has been deemed advisable, after careful consideration, to employ the same sieves for subgrade soil investigations as for the analysis of other highway materials, mainly because they are already a part of the equipment of highway testing laboratories, and because analyses made with them are more intelligible to the highway engineer.

The analyses published in this issue of PUBLIC ROADS, in the article entitled "Reinforcing and the subgrade as factors in the design of concrete pavements," were made with Tyler standard mesh sieves. It will be noted that the openings are defined by the number of meshes to the inch. The fractions designated as clay and silt are those which pass the 200-mesh sieve, and they are separated by subsidence in water according to a method which classifies particles whose diameter is 0.010 millimeter and less as clay and above that size as silt. The desirability of adopting one method for all subgrade analyses will be perceived if the reader will compare the above analyses with those published in the August issue of this magazine in the article entitled "Practical field tests for subgrade soils." The latter analyses were made with metric sieves according to the method followed by the Bureau of Soils.

It will be noted, for example, that in the important matter of determining the percentage of clay the two methods of analysis give different percentages because the particles designated as clay by the two methods have different maximum sizes. As stated in the footnote on page 4, the maximum size of the clay particles in the Bureau of Public Roads analysis is twice as great as the size of particles so designated by the Bureau of Soils method using metric sieves. It naturally follows that the percentage of clay indicated by the Bureau of Public Roads method is considerably greater than the Bureau of Soils method shows. The difficulty of comparing the results of the two methods is illustrated by the parallel grouping of the limits of the two methods in the next column.

Relation between metric sieves and Tyler standard mesh sieves

Analysis by metric sieves		Size of Tyler mesh	Analysis by Tyler standard mesh sieves	
Designation	Sieve size		Sieve No.	Designation
	Millimeters	*Millimeters*		
	2.000			
				Coarse material.
Fine gravel		1.651	10	
	1.000			
Coarse sand		0.833	20	Sand.
	0.500			
Medium sand				
	0.250			
Fine sand		0.221	60	
	0.100	0.147	100	
Very fine sand		0.074	200	
	0.050			
Silt		0.010		Silt.
	0.005			
Clay	0.000	0.000		Clay.

By plotting the mechanical analysis obtained by either method as a cumulative percentage curve it is possible to convert one analysis into terms of the other, although the values obtained are, of course, only approximate. The abscissae of such a curve would represent the sieve sizes in millimeters, and the ordinates cumulative percentages passing the respective sieves. The conversion is made by reading from the curve the cumulative percentages represented by the ordinates of the points at which the curve is cut by perpendiculars to the horizontal axis representing the

size limits of the other system of sieves. By subtracting the cumulative percentages thus obtained for each sieve from the corresponding percentages for the next larger sieves the percentage retained on each sieve is obtained.

The result of converting two of the analyses in Table 2, page 4, into the form in which they would appear if the analyses had been made with metric instead of with Tyler standard mesh sieves is shown below.

Mechanical analysis

Section No.	Station	Millimeters, per cent between							
		Over 2	2–1	1–0.5	0.5–0.25	0.25–0.1	0.1–0.05	0.05–0.005	0.005–0
		Coarse gravel	Fine gravel	Coarse sand	Medium sand	Fine sand	Very fine sand	Silt	Clay
1	28+00	22.5	3.8	10.0	10.7	11.4	6.1	22.5	13.0
2	30+00	17.0	4.0	11.5	12.5	17.7	7.3	20.0	10.0

The analyses published in this issue were made with Tyler sieves and these analyses have been used by way of example in the foregoing comparisons. These sieves are used at present by the Bureau of Public Roads laboratory, but the bureau does not wish to imply that the Tyler scale should be adopted as the standard for highway work. On the contrary, as stated in the April issue of PUBLIC ROADS, the bureau believes that the Bureau of Standards screen scale should be generally adopted and is prepared to conform to that scale as soon as it is accepted. In all likelihood it will ultimately be adopted by the American Society for Testing Materials and the American Association of State Highway Officials, and the bureau is hopeful that such action will be taken with as little delay as possible.

(Continued from page 15)

prone to take advantage of bidding of this sort and often widen cuts to secure extra material because, under the accepted bid, it is as cheap to do this as it is to borrow alongside and, in their opinion, better for the road. As a matter of fact, under an average unit price bid such as the above, the contractor is fairly certain to lose on the longer hauls involved in extra work built with material secured in the cuts. On the other hand, under this system of bidding the State pays too much for extra material secured from side borrow. Hence, it would seem to be better for every one concerned if a correct differentiation were made between excavation and borrow and if overhaul were bid in at a correct figure. The following indicates the method which may be pursued in making such a differentiation and shows the relationship which really exists between the cost of side-borrow and excavation on an ordinary fresno project. Overhaul is also calculated as a separate item, a 200-foot free haul (center to center of mass rule) being assumed. The details of the recast are not given.

Item	Unit	Amount	Time units	Days work for outfit of 10 fresnoes assumed	Bid price on work at $156.50 per day	Computed unit price bid
		Cu. yds.		*Days*		*Cents*
Borrow	Cubic yards.	35,277.2	59,971.2	30	$4,695	13.3
Excavation	....do.....	58,718.2	153,251.7	76.6	11,988	20.4
Overhaul	Cubic yards per station.	32,827.9	32,827.9	16.4	2,567	7.8
Total			246,050.98	123	19,250	

From the standpoint of the officials responsible for design, bids which differentiate between excavation and borrow should be of considerable value for they would at once place a large premium on careful design involving the avoidance of all unnecessary haul if fresnoes are to be used. From the standpoint of the contractor a bid of this kind is equally of advantage for while working under it field changes can not deprive him of his profit as sometimes happens when borrow and excavation are taken at the same price and the engineer decides to substitute excavation (wider cuts) for a large part of the borrow. This style of bidding is, then, of value to the State and is a real protection to the contractor's profits.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPT. 30.1924

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 575,007.24	$ 284,651.86	86.1	$ 15,630,308.90	$ 7,534,554.61	836.6	$ 79,416.15	$ 39,708.07	0.6	$ 1,207,800.92	Alabama
Arizona	8,338,365.41	4,287,683.88	527.8	713,173.87	406,825.70	37.5	1,600,010.00	968,451.46	127.3	222,983.72	136,265.33	32.7	1,696,474.63	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	122,739.41	50,850.57	4.9	7,923,828.33	2,920,437.11	394.2	544,200.78	239,497.48	49.2	1,427,269.21	Arkansas
California	12,999,075.03	5,647,148.17	533.7	2,715,077.91	1,473,319.61	90.2	12,759,091.73	6,642,729.58	439.3	435,211.04	256,774.51	1.3	3,073,334.13	California
Colorado	8,108,070.31	4,029,898.97	502.6	323,679.33	174,547.77	28.9	5,582,952.15	3,015,081.10	193.8	35,740.10	20,057.34	2.9	2,320,295.82	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	199,024.56	98,423.00	4.9	3,262,054.48	1,045,804.04	54.1				967,409.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	462,969.09	197,825.82	13.8	608,205.95	214,911.10	13.8	710,582.29	289,320.00	19.3	29,758.25	Delaware
Florida	961,134.07	461,470.92	48.8	441,141.73	220,570.86	12.8	9,225,920.22	4,498,526.04	248.7	398,642.71	171,123.94	10.3	935,195.24	Florida
Georgia	17,167,373.32	7,955,805.20	1214.2	534,460.21	265,330.76	46.2	9,828,434.90	4,879,632.73	757.1	1,784,431.08	843,312.03	108.9	505,816.28	Georgia
Idaho	8,181,697.92	4,092,395.52	506.8	197,306.67	113,420.29	18.7	1,644,869.99	949,544.66	118.4	809,078.31	533,904.13	50.8	988,447.41	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	663,460.35	327,523.92	20.6	16,436,813.06	8,167,813.86	658.5				2,661,607.89	Illinois
Indiana	7,577,444.16	3,655,640.97	225.7	1,182,752.58	581,613.20	41.2	15,951,610.62	7,770,338.46	517.5				2,304,899.37	Indiana
Iowa	23,195,778.19	9,237,031.86	1682.9	282,803.25	113,620.08	20.2	8,697,847.92	4,038,449.47	632.0	297,707.62	147,800.00	25.6	1,799,235.59	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	3,806,022.20	1,404,334.99	108.9	15,347,582.65	6,104,078.33	618.0	1,476,394.86	706,878.71	92.2	1,040,820.17	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	596,589.15	241,290.72	23.2	8,917,340.25	4,119,294.28	338.4	680,318.04	307,498.91	33.5	1,089,707.81	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	738,057.99	358,026.14	51.5	5,254,523.47	2,602,920.69	338.3	271,338.99	135,669.49	9.1	532,682.42	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	260,943.30	129,792.71	10.0	1,613,995.65	802,565.56	62.3	235,761.31	93,583.78	8.7	764,094.57	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	230,745.50	115,372.74	10.8	2,382,355.91	1,017,083.73	73.6	741,077.98	294,426.81	21.5	8,744.94	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	88,434.91	28,760.00	1.4	6,377,796.76	2,041,497.59	106.5	485,367.78	146,756.38	7.2	1,597,038.81	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5				14,428,493.46	6,913,576.62	507.5				2,905,583.15	Michigan
Minnesota	24,037,561.24	9,885,843.07	2292.0	675,006.59	304,379.76	53.2	9,973,348.31	4,631,929.97	842.3	443,530.96	196,400.00	80.2	299,866.20	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	357,385.12	178,692.55	18.0	8,235,805.97	4,103,716.98	493.2	601,622.36	299,873.30	43.9	1,120,048.78	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.5	592,423.54	295,591.65	49.2	18,273,683.60	8,752,448.06	766.2	3,797,210.31	1,775,686.46	196.3	1,870,562.45	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	108,869.05	87,529.84	12.4	2,884,950.84	2,115,151.46	268.2	1,085,404.96	615,745.03	122.2	3,763,655.56	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	200,802.81	100,401.40	22.6	7,694,728.23	3,751,494.73	788.4	1,028,632.63	514,316.26	120.0	3,370,042.02	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	587,270.26	511,310.71	43.3	4,870,499.36	4,077,547.69	412.6	168,117.37	146,631.94	19.3	301,205.68	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	20,801.55	10,308.46	1.1	1,443,234.25	678,912.21	47.3	138,590.95	67,471.46	4.6	190,404.27	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	298,751.09	111,360.00	5.6	9,724,981.71	2,795,603.75	73.4	796,997.14	205,677.49	10.6	815,074.27	New Jersey
New Mexico	5,306,286.45	2,758,849.68	714.3	381,468.92	231,065.57	51.3	6,620,900.78	4,348,083.59	656.8	389,438.39	239,504.59	27.7	1,011,828.57	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	468,489.72	191,631.63	14.5	29,534,066.96	11,183,066.72	659.6	5,016,794.00	1,427,433.75	92.9	5,648,171.46	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	241,113.23	112,077.78	12.5	11,200,653.19	4,208,856.02	316.5	1,719,594.43	807,790.46	39.9	1,488,769.08	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1587.9	376,098.20	187,597.07	81.0	3,641,820.37	1,785,327.54	565.6	184,426.85	92,213.39	20.8	1,880,012.58	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	1,205,099.59	468,814.13	30.3	13,210,878.97	5,014,387.60	364.1	2,187,800.71	741,115.00	66.8	2,035,929.28	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	1,187,137.45	572,302.34	52.8	8,234,521.73	3,959,190.42	364.5	2,269,348.24	998,272.23	99.3	1,118,085.98	Oklahoma
Oregon	12,082,873.17	6,819,093.79	655.6	513,232.52	303,539.04	48.5	3,347,304.72	1,980,045.65	181.6				403,480.52	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	415,284.23	176,961.99	9.7	22,662,779.55	6,306,421.25	381.1	2,673,574.86	742,641.25	48.7	3,260,896.72	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	157,932.08	62,600.00	3.1	1,243,889.16	438,306.80	25.2	380,178.15	92,700.00	6.2	560,206.24	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	824,054.97	372,624.85	142.2	5,783,649.52	2,402,577.79	423.0	603,658.77	190,330.15	57.7	597,967.99	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	845,706.40	453,412.58	120.0	7,426,924.78	3,701,926.79	1000.0	366,912.37	183,456.16	56.4	135,248.20	South Dakota
Tennessee	6,805,683.35	3,313,936.07	259.6	1,261,420.91	630,710.45	37.5	14,016,675.13	6,393,573.37	450.3	1,782,093.23	809,835.64	88.6	876,581.47	Tennessee
Texas	42,341,998.56	16,190,624.91	3122.8	2,121,853.15	951,671.72	166.0	25,591,436.99	10,134,114.56	1562.0	3,111,221.73	1,212,001.07	175.4	3,235,800.74	Texas
Utah	3,304,423.75	1,895,805.92	219.0	719,066.79	390,456.24	91.2	3,658,519.93	2,411,856.29	212.9	1,211,778.50	839,681.79	78.1	578,672.76	Utah
Vermont	1,922,114.16	942,769.12	74.4				1,999,561.44	956,178.98	50.8	165,057.47	73,677.20	5.1	561,353.70	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	312,421.07	140,321.52	8.9	9,242,088.09	4,299,993.56	323.7	1,238,244.82	476,332.88	37.6	874,522.61	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	89,400.58	42,436.47	11.2	3,720,136.20	1,697,300.00	146.0	594,429.73	283,000.00	20.0	573,046.08	Washington
West Virginia	5,489,747.95	2,365,041.53	255.6	898,083.14	402,583.56	24.7	4,428,254.78	1,998,238.67	155.6	528,260.11	232,245.33	18.6	756,022.91	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1325.3	604,737.13	291,645.40	31.1	4,363,817.49	2,181,492.74	184.6	219,299.22	66,905.70	7.0	3,697,373.59	Wisconsin
Wyoming	8,127,625.61	3,078,098.70	687.6	412,650.54	225,012.57	57.1	4,652,110.25	2,951,679.84	320.6	77,217.69	49,921.00	11.7	382,638.89	Wyoming
HAWAII													365,625.00	HAWAII
TOTALS	$ 549,655,391.27	$237,852,399.82	32452.9	$30,010,949.88	$ 14,393,140.24	1849.8	$ 411,055,258.70	$ 185,506,713.94	18971.9	$ 41,987,688.68	$ 17,743,436.44	2031.4	$ 69,629,309.56	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 66,150,590.24 Federal aid $ 30,561,567.30 Miles 3,259.7

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

REPORTS

Report of the Director of the Bureau of Public Roads for 1918.
Report of the Chief of the Bureau of Public Roads for 1919.
Report of the Chief of the Bureau of Public Roads for 1920.
Report of the Chief of the Bureau of Public Roads for 1921.
*Report of the Chief of the Bureau of Public Roads for 1922. 5c.
*Report of the Chief of the Bureau of Public Roads for 1923. 5c.

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220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
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*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
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59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
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*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
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74. State Highway Mileage and Expenditures for the Calendar Year 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D-3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

*Department supply exhausted.

Have you ever Stood By the Seashore?

HAVE YOU EVER STOOD BY THE SEASHORE and watched the progress of an ocean liner way off near the horizon? If you have you will remember that it scarcely seemed to move at all, although you were sure it was moving, and moving swiftly too. You recall that when you turned your eyes away for a time and then looked back you found it in an entirely different quarter.

PROGRESS IN THE IMPROVEMENT OF AMERICA'S HIGHWAYS is very much like the motion of that ship. The immensity of the task ahead makes our best efforts, somehow, seem poor and weak indeed. Although Federal-aid roads are being completed at the rate of nearly 10,000 miles a year and an equal mileage, perhaps, is built on the Federal aid system each year without Government assistance, the ultimate improvement of the main system and the necessary auxiliary roads seems sometimes discouragingly remote. When you feel that way just look back ten years and you will find in the tremendous improvement of American roads during that period the encouragement you need. There is now a definite program of construction in all States. Every road improved in accordance with the program forges another link in the system. Section by section the gaps in the projected network are filled in. Mile by mile the sections of improved roads draw toward each other; and one fine day we shall come to the realization that we have been building better and more swiftly than we knew.

ASK ANY OF THE STATES that stand near the head of the highway procession and they will tell you that there is only one opinion about highway improvement when the gaps begin to close.

A 22.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 9 ▽ NOVEMBER, 1924

GENERAL VIEW OF APPARATUS FOR STATIC LOAD TESTS OF CONCRETE SLABS, ARLINGTON, VA.

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1924

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 5, NO. 9 NOVEMBER, 1924 H. S. FAIRBANK, Editor

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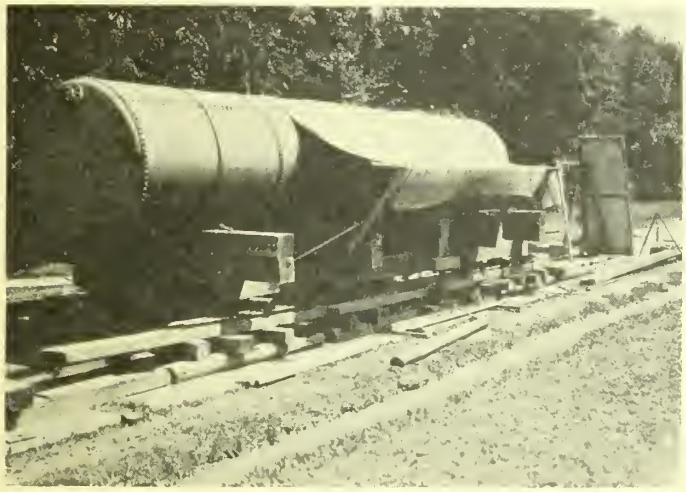
STATIC LOAD TESTS ON PAVEMENT SLABS

By J. T. THOMPSON, Highway Research Specialist, United States Bureau of Public Roads

FOUR years ago the Bureau of Public Roads began the testing of approximately 50 small typical road slabs under impact forces. The success of this early program led in the summer of 1921 to the construction of 124 larger slabs embracing a greater variation of type. These slabs were tested under impact during the summer of 1923 and the results of the test were fully reported in the article entitled "Impact tests on concrete pavement slabs," published in PUBLIC ROADS, volume 5, No. 2, April, 1924.

After these impact tests were concluded it was thought that a check on the impact results and additional information could be obtained by subjecting the slabs to tests under loads applied statically instead of dynamically. It is with these static tests, conducted during the summer of 1924, that the present report concerns itself.



The tank used in the static load tests, showing saddles and cribbing

SUMMARY OF CONCLUSIONS

1. The static resistance of both the corners and the edges of the rigid slabs is affected by the nature of the subgrade, the more resistant the subgrade to load the greater the resistance of the slab, and vice versa.

2. The resistance of the rigid slabs to static loads does not vary with the square of the depth but as some power greater than one and less than two. About 1.75 is the average value, the exponent being higher for slabs on the wet subgrade and lower for those on the dry subgrade.

3. The corners and edges of concrete slabs of the size and thickness tested offer about the same degree of resistance to static loads.

4. The presence of mesh reinforcement as employed in the slabs under consideration does not increase the load-carrying capacity of concrete slabs but does give rise to a tendency to hold together and resist complete failure after initial or elastic failure has taken place.

5. The bituminous topping laid on the rigid slabs does not increase the resistance of the slabs to static load.

6. At the ordinary summer temperatures encountered, bituminous slabs of the types tested show no slab strength.

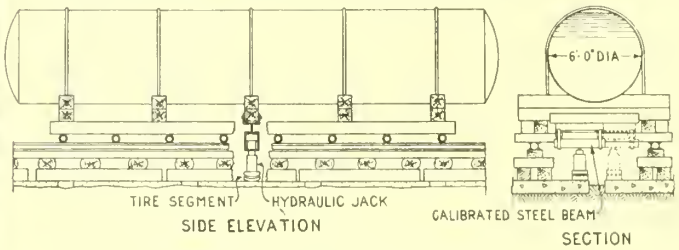
DESCRIPTION OF SLABS

Of the 124 slabs, 56 were placed on a carefully drained subgrade and 68 on a subgrade which was kept permanently saturated by water-filled ditches parallel to and on both sides of the line of slabs, the water standing level with the bottom of the slabs. The slabs were cast in duplicate and the more important types were laid on both the wet and the dry subgrades, which were carefully cut to the grade of the slab bottom to prevent the original structure of the soil from being disturbed.

In the mesh-reinforced slabs the reinforcement was placed uniformly 2 inches from the top, and where two layers were used they were placed in contact with each other.

The different types may be divided roughly into five groups:

1. Plain concrete.
2. Reinforced concrete.
   (*a*) Mesh-reinforced.
   (*b*) Rod-reinforced .[1]
3. Concrete bases with bituminous tops.



Arrangement of apparatus for static load tests

4. Bituminous bases with bituminous tops.
5. Macadam bases with bituminous tops.

A detailed description of the slabs is given in Table 1. For additional information relative to the construction of the slabs, the reader is referred to the article entitled "Impact tests on concrete pavement slabs," published in PUBLIC ROADS, volume 5, No. 2, April, 1924.

TABLE 1.—*Description of slabs—All slabs 7 feet square of material and thickness shown below—Series 200–267, on wet subgrade Series 300–377, on dry subgrade*

PLAIN CONCRETE SLABS AND BASES

Slab No.	Base course		Binder course		Surface course	
	Thickness	Material	Thickness	Material	Thickness	Material
	Inches		*Inches*		*Inches*	
200	6	Macadam	4	Bituminous concrete.	2	Topeka.
201	6	do	4	do	2	Do.
204	4	do	3	do	2	Do.
205	4	do	3	do	2	Do.
208	4	do	2	do	2	Do.
209	4	do	2	do	2	Do.
212	6	do			2	Do.
213	6	do			2	Do.
214	9	do			2	Do.
215	9	do			2	Do.
216	12	do			2	Do.
217	12	do			2	Do.
218	6	Bituminous concrete.			2	Do.
219	6	do			2	Do.
220	4	do			2	Do.
221	4	do			2	Do.

[1] Not tested statically.

16254—24†

TABLE 1.—*Description of slabs.—All slabs 7 feet square of material and thickness shown below. Series 200–267, on wet subgrade. Series 300–337, on dry subgrade.*—Continued

PLAIN CONCRETE SLABS AND BASES—Continued

Slab No.	Base course		Binder course		Surface course	
	Thickness	Material	Thickness	Material	Thickness	Material
	Inches		*Inches*		*Inches*	
222	6	1:3:6 concrete	1½	Bituminous concrete.	1½	Sheet asphalt.
223	6	do	1½	do	1½	Do.
224	6	do			1	Coarse bituminous concrete.
225	6	do			4	Do.
226	6	do			2	Do.
227	6	do			2	Do.
228	4	do			2	Topeka.
229	4	do			2	Do.
230	4	1:1½:3 concrete			2	Do.
231	4	do			2	Do.
232	6	1:3:6 concrete			2	Do.
233	6	do			2	Do.
234	6	1:1½:3 concrete			2	Do.
235	6	do			2	Do.
236	8	1:3:6 concrete			2	Do.
237	8	do			2	Do.
238	8	1:1½:3 concrete			2	Do.
239	8	do			2	Do.
240					4	1:1½:3 concrete
241					4	Do.
242					6	Do.
243					6	Do.
244					6	1:3:6 concrete
245					6	Do.
246					8	1:1½:3 concrete
247					8	Do.
248					8	1:3:6 concrete
249					8	Do.
300	4	Macadam	2	Bituminous concrete.	2	Topeka
301	4	do	2	do	2	Do.
304	4	do			2	Do.
305	4	do			2	Do.
306	6	do			2	Do.
307	6	do			2	Do.
308	9	do			2	Do.
309	9	do			2	Do.
310	6	Bituminous concrete.			2	Do.
311	6	do			2	Do.
312	4	do			2	Do.
313	4	do			2	Do.
314	6	1:1½:3 concrete			2	Do.
315	6	do			2	Do.
316	6	1:3:6 concrete			2	Do.
317	6	do			2	Do.
318	4	1:1½:3 concrete			2	Do.
319	4	do			2	Do.
320	8	do			2	Do.
321	8	do			2	Do.
322					4	1:1½:3 concrete
323					4	Do.
324					6	Do.
325					6	Do.
326					6	1:3:6 concrete
327					6	Do.
328					8	1:1½:3 concrete
329					8	Do.

CONCRETE SLABS WITH MESH REINFORCING—WET SUBGRADE

Slab No.	Surface course		Reinforcement	
	Thickness	Material	Description	Lbs. per sq. ft.
	In.			
250	4	1:1½:3 concrete	1 layer No. 6	0.30
251	4	do	do	.30
252	4	do	2 layers No. 6	.60
253	4	do	do	.60
254	1	do	2 layers No. 8	.75
255	4	do	do	.75
256	4	do	1 layer No. 10	.46
257	4	do	do	.46
258	4	do	2 layers No. 10	.93
259	4	do	do	.93
260	6	do	1 layer No. 6	.30
261	6	do	do	.30
262	6	do	2 layers No. 6	.60
263	6	do	do	.60
264	6	do	2 layers No. 8	.75
265	6	do	do	.75
266	6	do	2 layers No. 10	.93
267	6	do	do	.93

CONCRETE SLABS WITH MESH REINFORCING—DRY SUBGRADE

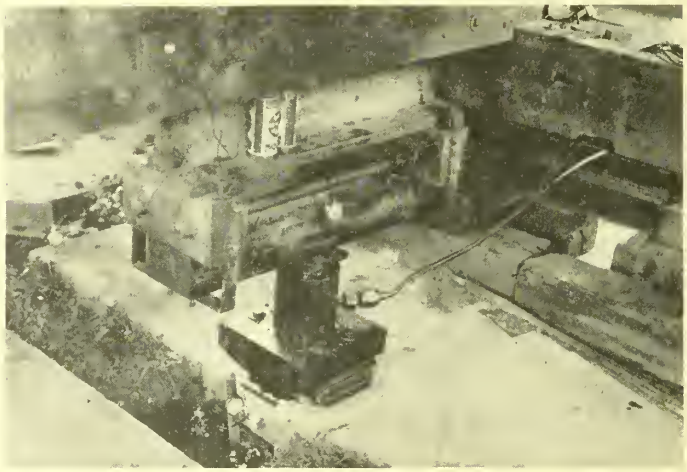
330	4	1:1½:3 concrete	1 layer No. 6	0.30
331	4	do	do	.30
332	4	do	1 layer No. 10	.46
333	4	do	do	.46
334	6	do	2 layers No. 6	.60
335	6	do	do	.60
336	6	do	2 layers No. 8	.75
337	6	do	do	.75

DESCRIPTION OF TEST METHODS AND APPARATUS

In the static tests the loads were applied to the slabs by a hydraulic jack through the medium of a segment of a solid rubber truck tire bolted to a cast-iron block. The necessary reaction for the jack was furnished by a large tank filled with water which the jack partially lifted as it applied load to the slab. This load was measured by a simply supported 4-inch by 6-inch chrome-nickel steel beam on a 30-inch span, the upward reaction of the jack loading this beam at the midpoint. The deflections of this beam, from which the loads were determined, were measured by an Ames dial, reading to ten-thousandths of an inch, which was mounted on the beam with its plunger resting on a one-half-inch square steel bar pivoted to the beam under the supporting knife edges at the neutral axis.

The deflections of the bituminous slabs were measured by engineers' scales set vertically on the tire block, the readings being made against horizontal arms secured to steel pins driven firmly into the subgrade. Two scales were employed and the average used to take account of the tilting of the tire block.

The deflections of the rigid slabs were measured by means of an Ames dial which recorded the movement



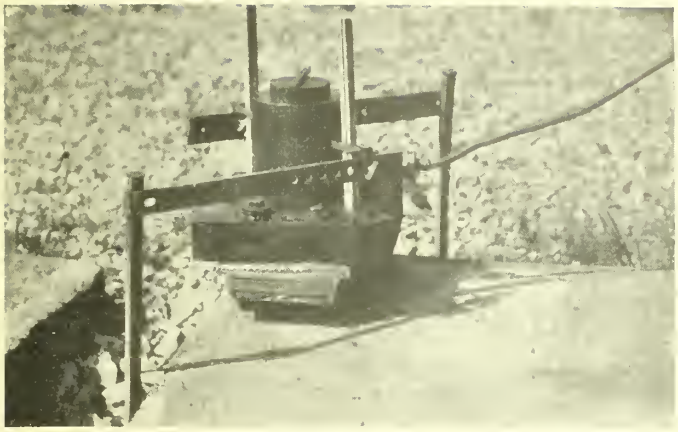
View of apparatus showing calibrated steel beam with ten-thousandths inch Ames dial measuring its deflection, hydraulic jack, tire block, slab deflection gauges at point of test and at corner, and graphic strain gauges in top and bottom of the slab at the edge

of a pin set into the corner or edge of the slabs, while the tilting of the slabs as a whole was recorded by other Ames dials mounted usually at the three corners of the slabs.

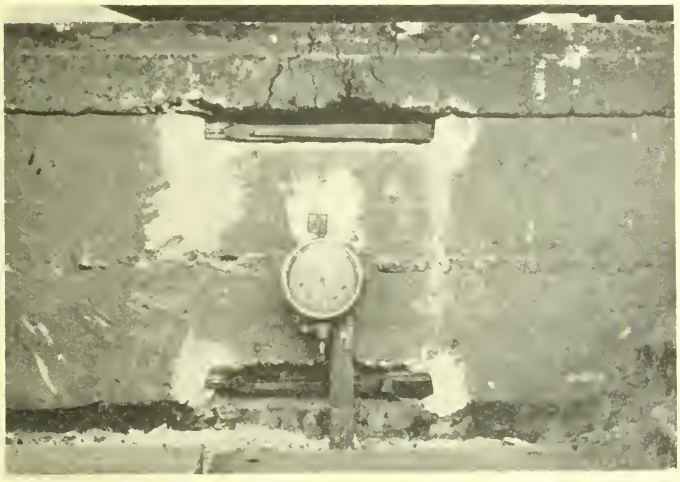
The fiber deformations produced were measured by means of graphic strain gauges of the type developed by the Bureau of Public Roads and described in Engineering News-Record, March 22, 1923, in an article entitled "A new impact strain gauge," by A. T. Goldbeck. In testing the edges such gauges were used in both the top and bottom of the slab. In testing the corners they were set along the diagonal on the top of the slab, one gauge overlapping its neighbor so as to insure at least one gauge catching the crack at failure and recording the maximum fiber stress.

In the edge tests the load was applied at the middle of the side of the slab and 5 inches from the edge. It was found that if the load were applied at a point closer to the edge the tire segment would overhang when expanded under load. In testing the corners the load was applied 7 inches from the corner measured along the diagonal of the slab.

The subgrade bearing value was determined by the method described in PUBLIC ROADS, volume 4, No. 5,



Tire block and jack with engineers' scales for measuring slab deflection against stationary arms



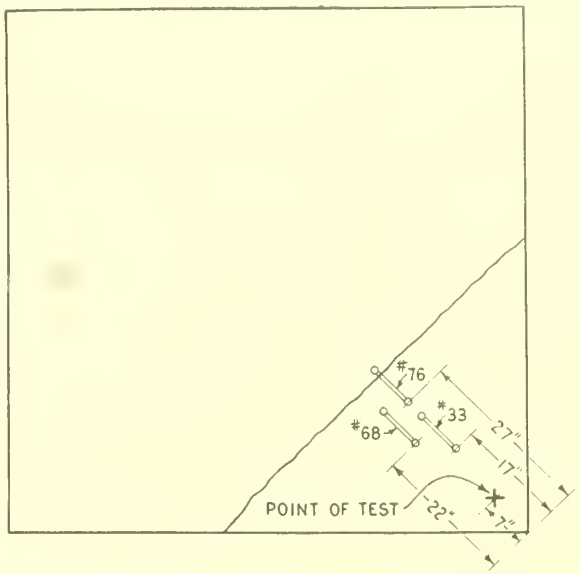
Graphic strain gauges arranged to measure compressive and tensile stresses in edge of slab; Ames dial measures deflection

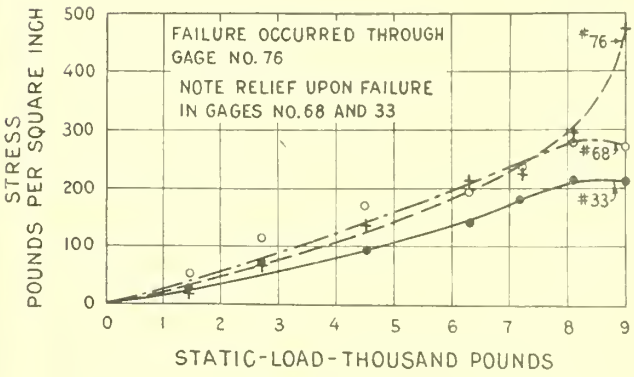
Three graphic strain gauges set along diagonal of slab, overlapping so as to catch maximum fiber stress

September, 1921, in the article entitled "Preliminary report on the Bates experimental road." The method of test was to apply a constantly increasing load to a small circular bearing block in contact with the scraped surface of the subgrade, and to measure the penetration of this foot into the subgrade at equal intervals of load.

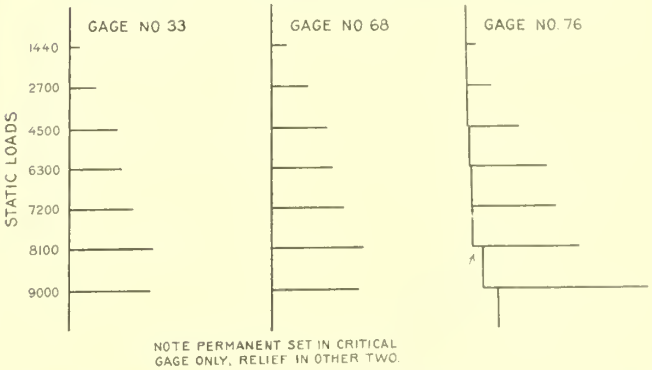
DISCUSSION OF DATA

Before discussing the test data certain explanatory remarks are necessary. The concrete fiber stresses were measured in the following manner: Records of the fiber deformations for various static loads were inscribed by the graphic strain gauge upon the smoked-glass plate which is a part of the instrument. These plates were then fixed with varnish, placed in a projecting lantern, and the greatly magnified records were measured. The measurements of total fiber deformation were reduced to strain. The unit stresses corresponding to these strains were computed from the modulus of elasticity as determined from tests on 6 by 12-inch cylinders which were cast at the time the slabs were constructed (1921) and were tested for modulus of elasticity at the time of





Corner test of slab No. 244, indicating static resistance of 8,100 pounds

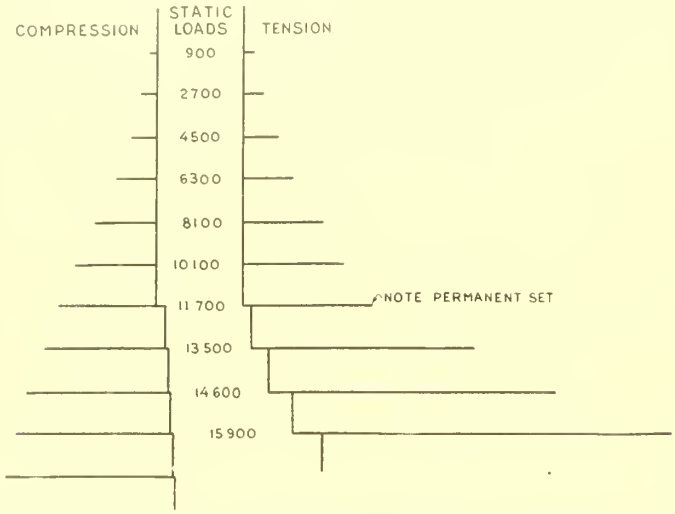


Magnified graphic strain gauge records of corner test of slab No. 244, indicating static resistance of 8,100 pounds

the impact tests (1923). Since they were two years old when the moduli were determined, it is felt that there was but little change during the ensuing year, and the 1923 values were therefore used without correction.

"Static load resistance" as used in this report is defined as that load which caused a tensile failure in the concrete. This tensile failure was detected by the graphic strain gauges. In the edge tests neutral axis

plots were made and the load which caused the neutral axis to shift upward was taken as the "static load resistance." The shifting of the neutral axis was taken as indication that the point of elastic failure had been reached. A study of the strain gauge records to see where large initial sets accompanied by rapidly increasing deformations began to appear, together with a study of curves plotted between static loads and unit fiber stresses, also helped to determine this point of initial failure. In the case of the corner tests, no gauges were placed in the edges and therefore no neutral-axis plots



Magnified graphic strain gauge records of edge test of slab No. 242, indicating a static resistance of 11,700 pounds

were made, the strain gauge records together with the load-stress plots furnishing the evidence to determine where failure occurred.

The point of elastic failure is in reality the ultimate load, as subsequent loads of an equal or even lesser amount cause complete failure. This was noted in several instances.

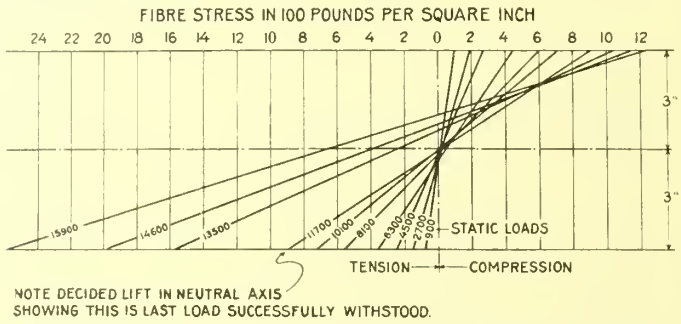
The data are somewhat restricted in quantity due to the fact that a number of the slabs were so badly broken by impact as to render them useless for static tests. This greatly interfered with the drawing of comparisons between duplicate types under different conditions.

EFFECT OF NATURE OF SUBGRADE ON STATIC RESISTANCE

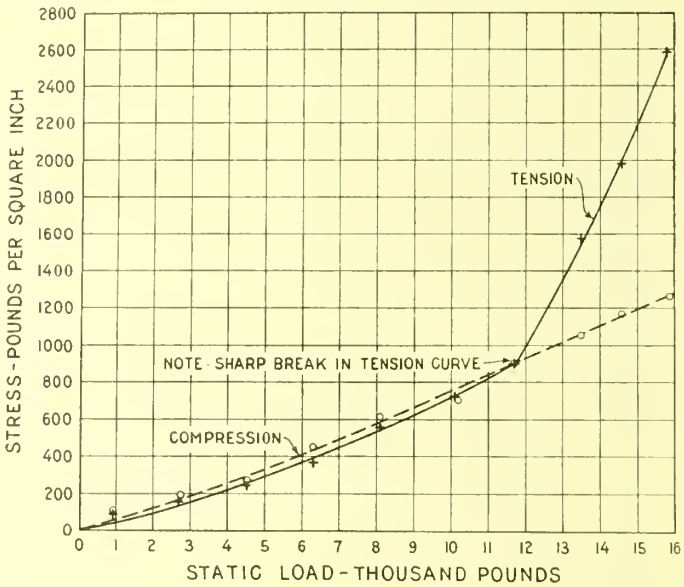
A study of Table 2 leads to the following conclusions:

1. That the slabs laid on a wet and yielding subgrade offer less resistance to statically applied loads than do those laid on a dry, firm subgrade.

2. That the resistance does not seem to vary in any definite manner with the subgrade bearing value as determined by the penetration test, the ratio of slab resistances being often high where the ratio of subgrade penetrations is low, and vice versa.



Stress distribution diagram for edge test of slab No. 242, indicating a static resistance of 11,700 pounds



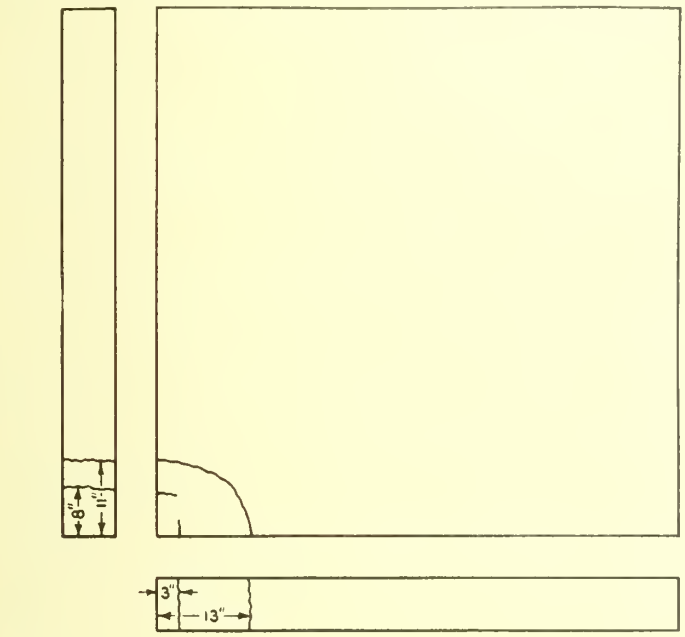
Load-stress diagram of edge test, slab No. 242, indicating static resistance of 11,700 pounds

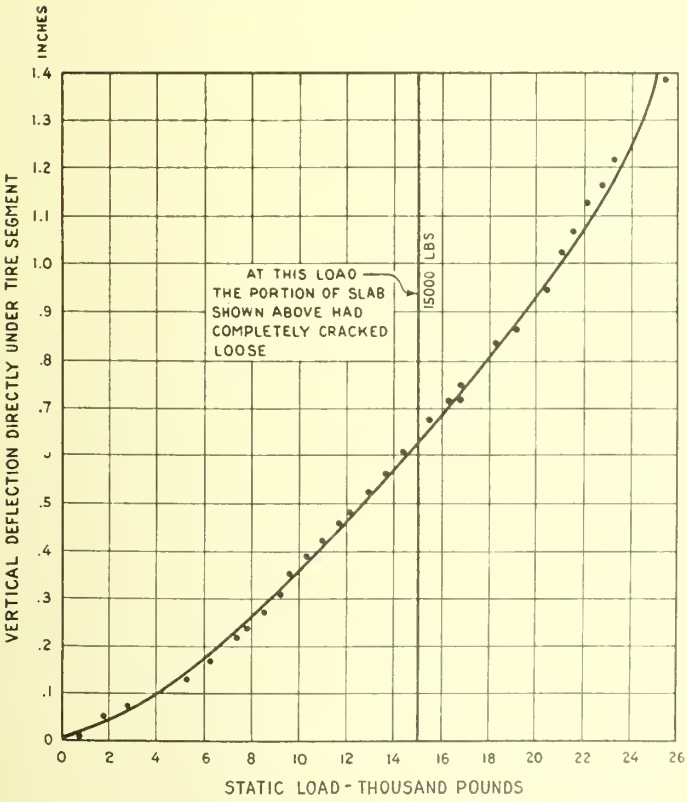
TABLE 2.—*Effect of nature of subgrade on static resistance*

Slab No.	Description of slab: Thickness	Mix	Reinforcement	Top	Position of load	Static resistance: Dry subgrade	Static resistance: Wet subgrade	Ratio of resistance dry-wet	Subgrade penetration at 100-pound load: Dry	Subgrade penetration at 100-pound load: Wet
	Inches					*Pounds*	*Pounds*		*Inches*	*Inches*
331	4	1:1½:3	1 layer No. 6 (0.30 pound per sq. ft.)		Corner	7,200		1.33	0.017	
251	4	1:1½:3	do		do		5,400			0.048
330	4	1:1½:3	do		Edge	8,460		1.88	.016	
250	4	1:1½:3	do		do		4,500			.045
333	4	1:1½:3	1 layer No. 10 (0.46 pound per sq. ft.)		Corner	7,200		1.80	.005	
257	4	1:1½:3	do		do		4,000			.049
332	4	1:1½:3	do		Edge	7,380		1.52	.017	
256	4	1:1½:3	do		do		4,860			.045
414	6	1:1½:3		2-inch Topeka	Edge	12,000		1.80	.019	
234	6	1:1½:3		do	do		11,160			.075
326	6	1:3:6			Corner	10,400		1.29	.007	
244	6	1:3:6			do		8,100			.041
335	6	1:1½:3	2 layers No. 6 (0.60 pound per sq. ft.)		Corner	15,300		1.28	.017	
263	6	1:1½:3	do		do		12,000			.033
334	6	1:1½:3	do		Edge	15,480		1.55	.025	
262	6	1:1½:3	do		do		10,000			0.029
321	8	1:1½:3		2-inch Topeka	Corner	20,000		0.99	.014	
239	8	1:1½:3		do	do		20,200			.016
320	8	1:1½:3		do	Edge	28,600		1.79	.011	
238	8	1:1½:3		do	do		16,000			.041
329	8	1:1½:3			Corner	22,000		0.97	.018	
247	8	1:1½:3			do		22,800			.097
328	8	1:1½:3			Edge	17,000		0.68	.012	
246	8	1:1½:3			do		25,200			.062

VARIATION OF STATIC LOAD RESISTANCE WITH SLAB DEPTH

Table 3 shows that the load resistance varies as some power of the slab depth which is greater than one and less than two, the average value of the exponent being about 1.75. The influence of the subgrade resistance upon the exponent is quite marked, the value of the exponent being 1.92, or quite near the square, on the wet, yielding subgrade and 1.5 on the dry subgrade.





Corner test of slab No. 311, 6-inch bituminous base with 2-inch Topeka top. Air temperature 81° F., slab temperature 77° F. Note that there is no sharp break of the curve at failure

TABLE 3.—*Variation of static load resistance with slab depth*

Depth of slab (all types included)	Corner tests		Edge tests	
	Average static resistance	Power of depth with which resistance varies	Average static resistance	Power of depth with which resistance varies
Dry subgrade:	*Pounds*		*Pounds*	
4 inches	7,430		7,980	
6 inches	15,300	1.78	13,720	1.33
8 inches	21,000	1.50	22,800	1.52
Wet subgrade:				
4 inches	5,400		5,720	
6 inches	11,700	1.90	12,500	1.93
8 inches	21,500	2.00	20,600	1.85

COMPARISON OF EDGE AND CORNER STATIC LOAD RESISTANCE

Table 4 compares the static resistance of the corners and edges of duplicate slabs and shows that there is no decided difference between them.

TABLE 4.—*Comparison of edge and corner static load resistance*

Slab No.	Description of slab				Position of load	Static resistance
	Thickness	Mix	Reinforcement	Top		
Dry subgrade:	*Inches*					*Pounds*
319	4	1:1½:3		2-in. Topeka	Corner	7,900
318	4	1:1½:3		do	Edge	8,100
331	4	1:1½:3	1 layer No. 6 (0.30 pound per sq. ft.).		Corner	7,200
330	4	1:1½:3	do		Edge	8,460
333	4	1:1½:3	1 layer No. 10 (0.46 pound per sq. ft.).		Corner	7,200
332	4	1:1½:3	do		Edge	7,380
335	6	1:1½:3	2 layers No. 6 (0.60 pound per sq. ft.).		Corner	15,300
334	6	1:1½:3	do		Edge	15,480
321	8	1:1½:3		2-in. Topeka	Corner	20,000
320	8	1:1½:3		do	Edge	28,600
329	8	1:1½:3			Corner	22,000
328	8	1:1½:3			Edge	17,000
Wet subgrade:						
241	4	1:1½:3			Corner	6,900
240	4	1:1½:3			Edge	8,100
251	4	1:1½:3	1 layer No. 6 (0.30 pound per sq. ft.).		Corner	5,400
250	4	1:1½:3	do		Edge	4,500
255	4	1:1½:3	2 layers No. 8 (0.75 pound per sq. ft.).		Corner	5,800
254	4	1:1½:3	do		Edge	6,120
257	4	1:1½:3	1 layer No. 10 (0.46 pound per sq. ft.).		Corner	4,000
256	4	1:1½:3	do		Edge	4,860
259	4	1:1½:3	2 layers No. 10 (0.93 pound per sq. ft.).		Corner	5,400
258	4	1:1½:3	do		Edge	5,040
235	6	1:1½:3		2-in. Topeka	Corner	9,900
234	6	1:1½:3		do	Edge	11,160
243	6	1:1½:3			Corner	11,900
242	6	1:1½:3			Edge	11,700
261	6	1:1½:3	1 layer No. 6 (0.30 pound per sq. ft.).		Corner	10,500
260	6	1:1½:3	do		Edge	13,320
263	6	1:1½:3	2 layers No. 6 (0.60 pound per sq. ft.).		Corner	12,000
262	6	1:1½:3	do		Edge	10,000
265	6	1:1½:3	2 layers No. 8 (0.75 pound per sq. ft.).		Corner	14,000
264	6	1:1½:3	do		Edge	14,400
267	6	1:1½:3	2 layers No. 10 (0.93 pound per sq. ft.).		Corner	12,000
266	6	1:1½:3	do		Edge	14,400
244	6	1:3:6			Corner	8,100
245	6	1:3:6			Edge	10,800
239	8	1:1½:3		2-in. Topeka	Corner	20,200
238	8	1:1½:3		do	Edge	16,000
247	8	1:1½:3			Corner	22,800
246	8	1:1½:3			Edge	25,200
237	8	1:3:6		2-in. Topeka	Corner	8,500
236	8	1:3:6		do	Edge	17,640

EFFECT OF QUANTITY OF MESH REINFORCEMENT ON STATIC LOAD RESISTANCE

It can be seen from Table 5 that the amount of mesh reinforcement employed in these slabs has apparently no influence upon either corner or edge static load resistance, slabs without reinforcement or with small amounts frequently developing greater resistances than those more heavily reinforced.

The mesh reinforcement did, however, tend to hold the slabs together after elastic failure had taken place and brought about a much less rapid failure when complete destruction occurred.

TABLE 5.—*Effect of quantity of mesh reinforcement on static load resistance*

Slab No.	Description of slab			Reinforcement	Position of load	Static resistance
	Thickness	Mix	Top			
Dry subgrade:	*Inches*					*Pounds*
319	4	1:1½:3	2-in. Topeka	None	Corner	7,900
331	4	1:1½:3	----	1 layer No. 6 (0.30 pound per sq. ft.)	do	7,200
333	4	1:1½:3	----	1 layer No. 10 (0.16 pound per sq. ft.)	do	7,200
318	4	1:1½:3	2-in. Topeka	None	Edge	8,100
330	4	1:1½:3	----	1 layer No. 6 (0.30 pound per sq. ft.)	do	8,460
332	4	1:1½:3	----	1 layer No. 10 (0.16 pound per sq. ft.)	do	7,380
314	6	1:1½:3	2-in. Topeka	None	Edge	12,000
334	6	1:1½:3	----	2 layers No. 6 (0.60 pound per sq. ft.)	do	15,480
Wet subgrade:						
241	4	1:1½:3	----	None	Corner	6,900
251	4	1:1½:3	----	1 layer No. 6 (0.30 pound per sq. ft.)	do	5,400
253	4	1:1½:3	----	2 layers No. 6 (0.60 pound per sq. ft.)	do	4,850
255	4	1:1½:3	----	2 layers No. 8 (0.75 pound per sq. ft.)	do	5,800
257	4	1:1½:3	----	1 layer No. 10 (0.46 pound per sq. ft.)	do	4,000
259	4	1:1½:3	----	2 layers No. 10 (0.93 pound per sq. ft.)	do	5,100
240	4	1:1½:3	----	None	Edge	8,100
250	4	1:1½:3	----	1 layer No. 6 (0.30 pound per sq. ft.)	do	4,500
254	4	1:1½:3	----	2 layers No. 8 (0.75 pound per sq. ft.)	do	6,120
256	4	1:1½:3	----	1 layer No. 10 (0.46 pound per sq. ft.)	do	4,860
258	4	1:1½:3	----	2 layers No. 10 (0.93 pound per sq. ft.)	do	5,040
243	6	1:1½:3	----	None	Corner	11,900
261	6	1:1½:3	----	1 layer No. 6 (0.30 pound per sq. ft.)	do	10,500
263	6	1:1½:3	----	2 layers No. 6 (0.60 pound per sq. ft.)	do	12,000
265	6	1:1½:3	----	2 layers No. 8 (0.75 pound per sq. ft.)	do	14,000
267	6	1:1½:3	----	2 layers No. 10 (0.93 pound per sq. ft.)	do	12,000
242	6	1:1½:3	----	None	Edge	11,700
260	6	1:1½:3	----	1 layer No. 6 (0.30 pound per sq. ft.)	do	13,320
262	6	1:1½:3	----	2 layers No. 6 (0.60 pound per sq. ft.)	do	10,000
261	6	1:1½:3	----	2 layers No. 8 (0.75 pound per sq. ft.)	do	14,400
266	6	1:1½:3	----	2 layers No. 10 (0.93 pound per sq. ft.)	do	14,400

EFFECT OF TOPPING COURSE IN STATIC LOAD RESISTANCE

A comparison of the static load resistances of similar slabs with and without bituminous topping courses is shown in Table 6. The table brings out the fact that the addition of the topping course does not increase the strength. Indeed it might be said that the reverse is true in some instances. Certainly the 1:3:6 slabs with bituminous tops on the wet subgrade are badly disintegrated, while the uncovered 1:3:6 slabs on the same subgrade are in fair condition. Furthermore the covered 1:3:6 slabs on the dry subgrade are apparently nearly as good as the uncovered ones. The 1:1½:3 concrete in the covered slabs on both the wet and dry subgrade compares favorably with that in the uncovered slabs.

It seems probable that this weakness is the result of the saturated condition of the lean concrete, surface evaporation being prevented by the bituminous covering.

TABLE 6.—*Effect of topping course upon static load resistance*

Slab No.	Description of slab			Position of load	Static resistance
	Thickness	Mix	Top		
Dry subgrade:	*Inches*				*Pounds*
316	6	1:3:6	2-inch Topeka	Corner	8,100
317	6	1:3:6	---do---	do	7,000
326	6	1:3:6	None	do	10,410
321	8	1:1½:3	2-inch Topeka	do	20,000
329	8	1:1½:3	None	do	22,000
320	8	1:1½:3	2-inch Topeka	Edge	28,600
328	8	1:1½:3	None	do	17,000
Wet subgrade:					
235	6	1:1½:3	2-inch Topeka	Corner	9,900
243	6	1:1½:3	None	do	11,900
234	6	1:1½:3	2-inch Topeka	Edge	11,160
242	6	1:1½:3	None	do	11,700
239	8	1:1½:3	2-inch Topeka	Corner	20,200
247	8	1:1½:3	None	do	22,800
238	8	1:1½:3	2-inch Topeka	Edge	16,000
246	8	1:1½:3	None	do	25,200
237	8	1:3:6	2-inch Topeka	Corner	8,500
249	8	1:3:6	None	do	12,600

STATIC TESTS OF BITUMINOUS SLABS

The corners and edges of 12 bituminous slabs Nos. 300 and 301 and 304 to 313, inclusive, were tested. It was apparently impossible to determine when the slabs failed, as in no instance did the load-deflection curves show a sharp break, notwithstanding the fact that loads were applied and curves plotted far beyond the point where wide cracks had opened up around the tire segment. In some instances the test was carried to a point where the section of slab directly under the tire segment was completely detached from the main slab and yet the load-deflection curves did not indicate a sharp break. The test was nothing more than a soil-penetration determination. The effect of these slabs seems to be merely to distribute the loads to some extent over the subgrade.

In view of the apparent impossibility of telling when the slabs failed, the test data for these 12 slabs are not included in this report; but the writer concludes from his observation that these bituminous sections, at least at the temperature encountered, have an insignificant, if any, slab strength.

THE CONSTITUTIONALITY OF MOTOR VEHICLE LICENSE FEES AND THE GASOLINE TAX

By HENRY R. TRUMBOWER, Economist, United States Bureau of Public Roads

SOON after automobiles made their appearance on the highways and their numbers began to increase appreciably, State after State inaugurated systems of licensing motor vehicles and charging specific annual fees. The license fees in the beginning were nominal and generally were calculated only to cover the administrative expenses and the inspection costs necessary in the enforcement of all such regulatory laws. Nevertheless it was not long before the legality and constitutionality of these automobile registration and license laws were tested in a number of State court proceedings.

New Jersey was one of the first States to pass a motor vehicle registration law and soon after its passage its supreme court was invoked to pass upon the constitutionality of this enactment. The act was passed by the legislature in 1905 and was entitled "An act defining motor vehicles and providing for the registration of the same and uniform rules regulating the use and the speed thereof." It provided, in addition to certain traffic regulations, that the applicant for a license must make a written statement containing his name, address, description and character of automobile, name of maker, manufacturer's number, and horsepower, and pay a registry fee of $1.

The single question presented to the court was whether these provisions were constitutional. (Unwen *v.* State, 44 Vroom. (N. J.) 530.) In approaching this question the court stated that the manner in which public streets and highways should be used had been the subject of frequent legislation, that the control of such use had often been delegated by the legislature, and that such legislation was considered essential to the safety and comfort of those who use the public highway.

In developing its line of reasoning the court cited as precedents cases which dealt with regulations respecting the use of streets by street railway companies. (North Hudson County Railway *v.* Hoboken, 12 Vroom 71; Trenton Horse Railroad Company *v.* Trenton, 24 Vroom 132; Cape May Railroad Company *v.* Cape May, 30 Vroom 393.) Pedestrians, drivers, and occupants of horse-drawn vehicles using the same streets, the court said, were faced with new dangers upon the introduction of heavy cars propelled by electricity and capable of great speed. For the purpose of protecting the former users of the highway regulations were made which limited the speed of these cars among other things, and also required that licenses be obtained for each car. Such regulations were upheld by the courts and were regarded as legitimate extensions of the State's police power.

In turning from the electric street car to the automobile it was observed that motor vehicles operated over the highways of the State were being propelled, or capable of being propelled, at even greater speed than street cars. The menace of these machines, driven by reckless, inexperienced, and incompetent persons, to all persons using the highways was obvious to everyone. The court considered the right of the legislature to protect other users of the State's streets and roads against the dangers accruing from the operation of the automobile as much justified as those regulations which restricted the operation of the street car. Those provisions of the automobile law which limited the speed and which required the display of lights and the sounding of signals by automobile operators were considered necessary in order to protect the general public in its use of the highways.

These requirements, in the opinion of the court, would be useless if those violating them could not be detected and punished. The license and registration features were provided so as to aid to the identification of the vehicles and of those responsible for their operation. The law was attacked on the score that the required license fee of $1 was a tax upon the automobile and thus constituted a species of double taxation which was considered unconstitutional by the opponents of the law.

NEW JERSEY COURT HOLDS LICENSE FEE IS NOT A PROPERTY TAX

The court held that this license fee could not be regarded as a tax upon property. It was not imposed upon the vehicle as such but upon the use of the vehicle upon the public roads. It was a settled question, as pointed out by the court, that under the police power a license fee could be imposed which did not exceed the necessary expense of issuing the license and of carrying on the work and activities incidental to the inspection and regulation of motor-vehicle operation. In the eyes of the court this kind of automobile license legislation did not in any way conflict with the provisions of the Constitution of the State of New Jersey nor with any section of the Federal Constitution. It has been argued that legislation of this character by a State was an interference with interstate commerce and that it denied to nonresidents the equal protection of the laws. The court stated that the only question which the Federal courts would consider in dealing with legislation of this character was whether the regulations with respect to the operation of motor vehicles were within the legitimate exercise of a State's police power.

The Federal Supreme Court had taken a very decided stand on this question in the Slaughterhouse cases (16 Wall. 36). In other States where similar issues were presented to the courts the legality and constitutionality of automobile license laws were upheld as an exercise of the police power. The contestants in all of these cases were evidently satisfied with the views and findings of the State courts because the issue was not presented to the United States Supreme Court at this time even though in a number of instances nonresident automobile owners were protesting the legality of such license measures passed by the several States.

The nature of the evolution of automobile registration and regulatory statutes passed by many of the States, as the number of motor vehicles kept on increasing and the problems of highway development and maintenance began to emerge, is again illustrated by amendments made to the New Jersey law and by the opinion of the supreme court upholding its constitutionality and reasonableness. In 1908, three years after the first law was passed, New Jersey raised its annual registration fee to $3 for registering an automobile of less than 30 horsepower and $5 for each

automobile of 30 horsepower or more, and fees were provided for licenses issued to drivers. This new legislation also provided that nonresidents would be required over their own signatures to designate the secretary of state as their agent within the State upon whom service could be made in case they were charged with a violation of any of the regulatory provisions of the act growing out of the operation of their cars over the streets and highways of the State.

Although the State's right to require the registration of motor vehicles and to exact a license fee had been upheld as an exercise of the police power, the validity of this new legislation was again attacked in the courts on the following grounds: First, because a tax was imposed upon automobiles not according to their true value but according to the horsepower of each automobile; second, that it was a system of double taxation in that the local assessor had already levied a tax upon this kind of property; third, that automobiles were singled out and put into a special class for purposes of taxation and that other kinds of property of similar character were excluded; and, fourth, that the requirement that nonresident owners and operators of motor vehicles must designate an agent upon whom legal processes could be served was a discrimination against citizens of other States and that the tax was a burden upon interstate commerce and a violation of the fourteenth amendment of the Federal Constitution. (Cleary *v.* Johnston, 50 Vroom (N. J.), 49.)

THE USE OF LICENSE FEES FOR ROAD MAINTENANCE QUESTIONED

It was contended by the objectors to this new act that its validity could not be sustained on the same grounds upon which the legality of the former act had been upheld because of the increase in the amount of the fees charged for registration, and that it had been transformed from a regulatory measure into a revenue measure for the reason that this revised enactment provided that the money thus collected and not needed in the administration of the law should be used as a fund for the repair of improved roads. The surplus funds were to be apportioned by the State treasurer among the counties of the State "according to the mileage of improved roads in each county, to be used for the repair of said roads." While the court did not definitely pass upon the power of the State to collect fees from automobiles which were to be used for highway purposes, it intimated that if such fees were imposed for revenue purposes the legislature had probably not exceeded its powers in that regard. The rules laid down in the previous case, in the opinion of the court, disposed of the constitutional objections raised against this act as a regulatory measure. The revenue aspects were considered more fully in a subsequent opinion where that issue was pressed more seriously. At the time when this case was decided the court suggested that the facts introduced into the record to show that the fees which were collected actually exceeded all of the costs incurred by the State and the local authorities in the administration of the law in all of its regulatory aspects were not sufficient to establish the fact.

So far as the objections were concerned which were urged by nonresidents, the court pointed out that, while the right of a nonresident to travel over the highways of the State was more than a privilege, the nonresident did not have an absolute right to the use of the highway. The restrictions in the law were largely safety measures, the court said, and applied to both residents and nonresidents alike. According to the court, the inherent speed of the automobile made it practically impossible to enforce the operating restrictions as to speed and mode of operation except through penalties imposed after infraction of such rules. The provision that nonresidents in taking out licenses for operating automobiles within the State give power of attorney to the secretary of state to accept legal service was a device for bringing nonresident violators of the State's automobile regulations before the courts of the State for trial in the same manner as residents of the State were tried for similar offenses. In the light of these facts it could not be considered that there was an interference with their constitutional rights. Both residents and nonresidents were placed upon the same plane of legal action.

THE USE OF LICENSE FEES FOR ROAD PURPOSES UPHELD

This pronouncement of the supreme court still did not satisfy the opponents of this particular law and they succeeded in bringing a second case involving the validity of the 1908 motor vehicle tax law, this time stressing the fact that the motor vehicle license tax yielded a very substantial sum over and above the administrative costs and claiming that on that account it was invalid and constituted in fact a burden upon interstate commerce in defiance of the Federal Constitution. The State supreme court upon further consideration of this question held that the imposition of license fees for revenue purposes was clearly within the sovereign power of the State. (Kane *v.* New Jersey, 81 (N. J.) 594.)

When it was shown in the record that the receipts derived from the automobile license taxes were so large that a considerable part of them were used for highway purposes the court took the position that since the State had spent large sums of money on improved highways, the facilities of interstate and intrastate commerce, it was entitled to make a charge and exact a remuneration for the maintenance of such public works. The theory of the automobile license tax as to its purpose was thus extended to include revenues for highway purposes in addition to a collection of funds barely sufficient to meet the expenses of administering regulations ordained and established in the beginning as a part of the State's police power. The license tax was thus made to serve a dual purpose.

In reviewing this matter, the court indicated that in former times the State had created agencies such as turnpike companies to construct and improve specified sections of highways and had given them authority to make certain charges for their use. What the State had done along these lines indirectly it could do directly, and the license tax could rightly be regarded as a fee for the use of the roads built wholly or in part by the State or by any of its political subdivisions. It had been observed that the rapidly moving motor vehicles brought about a rapid deterioration of the road surface and made necessary high maintenance expenditures. The motor vehicle tax law imposed upon the owners of all these vehicles driven over the roads a charge for the purpose of highway maintenance and upkeep. The court saw no objection to the State charging a lump sum for this use instead of a toll or mileage fee, and a classification of automobiles according to horsepower, in the opinion of the court, reflected to a certain degree the extent of the use and could be regarded as a reason-

able classification. The legislature had prescribed a reasonable mode of measurement. The conclusions of the court are stated in the final paragraph:

> The imposition is a license or privilege tax charged in the nature of compensation for the damage done to the roads of the State by the driving of these machines over them, and is properly based not upon the value of the machine but upon the amount of destruction caused by it.

When Michigan attempted to add the revenue or tax feature to its motor vehicle registration law the supreme court refused to sustain the new legislation on constitutional grounds which were more or less technical in character. In 1913 the legislature passed an amendment to the motor vehicle administration act of 1909 which provided for a license fee graduated according to horsepower instead of a flat fee. It was estimated that under the new law the revenues per motor vehicle would average about $12; under the former law a flat fee of only $3 per vehicle was collected when the license was issued. The amendment also provided that the revenues accruing from these new license fees should be devoted to highway purposes and that automobiles should thereafter be exempted from all property tax assessments. The supreme court held that the new act was a revenue act and that the amount of fees derived under this licensing law was far in excess of the necessary expenses of registration and administration. While the police-power feature was still a part of this act, it was held that the tax feature had been introduced through the advance in the fees and that in doing so the title of the original act, which made no provision for the taxation of motor vehicles, had been retained by the amendatory act. On the ground that the legislative procedure had been irregular the new law was held invalid. (Vernor *v.* Secretary of State, 179 Michigan, 157.)

LICENSE FEES HELD TO BE PRIVILEGE TAXES—NOT PROPERTY TAXES

At the next session of the legislature (1915) the act was passed again, this time clearly indicating in the title that it was for the purpose of raising revenues as well as an exercise of the police power. This new act was promptly appealed to the supreme court but the schedule of license fees was duly upheld. (Jasnowski *v.* Board of Assessors, 191 Michigan, 287.) The title of the new act was as follows:

> An act to provide for the registration, * * * regulation of motor vehicles, * * * and to provide for levying specific taxes upon such vehicles, * * * and to provide for the disposition of such funds, * * * and to exempt from all other taxation such motor vehicles so specifically taxed * * *.

Among the objections to the act there was the contention that the title provided for both regulation and taxation, two distinct objects. The court stated that whether or not the act should be declared invalid depended upon the nature of the tax which this new law imposed. In settling this question it had to be determined whether the new tax was in the nature of a property tax or a privilege tax. If it could be regarded as a property tax the court intimated that the contention of the objectors was well founded. Granting that the legislature, in the exercise of the police power, could enact a regulatory statute in which regulation and taxation were so blended as to have but a single purpose, the court concluded this tax was intended as a privilege tax rather than a property tax and that the technical objection to the form and substance of the act was not well taken.

The contention was also made that the legislature exceeded its power when it exempted the automobiles operated on the highways from the regular ad valorem tax and substituted for it this new scale of motor vehicle license fees. Here the court admitted that it was within the province of the legislature to determine what class of property should be taxed and what class should not be taxed except where there was an interference with provisions of the Constitution. The legislature had exercised its discretionary power; the court could not say whether this was done wisely or justly.

The final objection that was made was that the act provided for the collection of money by taxation of owners of vehicles in cities and villages to be expended on the rural highways. The court, in answer to this, pointed out that under the constitution the State was authorized to engage in works of internal improvement. The construction and maintenance of highways were considered to be works of this class. This constitutional provision, or rather permission, therefore, furnished the basis for the action of the legislature in disbursing the fund collected from automobile operators; and the legislature had the power to determine which roads should be improved first, knowing that not all the streets and roads of the State could be improved at one and the same time.

It is interesting to note in this connection that the court refrained from advancing the toll-road theory in trying to justify the collection of fees from persons owing motor vehicles and residing in cities and villages which did not benefit directly in having any of these highway funds allocated for the improvement of their streets. The argument that these city dwellers used the rural highways probably as much as the rural population and could therefore be charged for such use was not presented. In other cases this reason is considered of great weight.

HIGH MOTOR VEHICLE FEES MUST BE PREDICATED ON NEED OF HIGHWAY REVENUE

That the legislature must clearly predicate high motor vehicle fees upon the theory that such revenues are desired for highway purposes and that in the passage of such an act the revenue feature must be clearly brought out is exemplified in Alabama's attempt to increase its motor vehicle fees from a nominal sum charged for licenses to rather substantial fees based on horsepower. The act passed in 1913 was presented to the supreme court for judicial review and was declared unconstitutional. (State *v.* Lawrence, 61 South. 975 (Alabama).) The attorney general in presenting his case to the court maintained that the statute was not a revenue measure but a police measure pure and simple; that the matter of taxation was not involved. The court held that the State had, without any doubt, the right to regulate the use of its highways and that in doing so it could compel the registration and numbering of automobiles; that it could impose fees which would compensate the State for the expenses and costs which such legislation entailed, but that such fees had to be reasonable and fair considering the object to be attained and all the surrounding facts. Inasmuch as the fees varied from $5 to $25 and the registration expense was estimated to be not over $1, the court con-

cluded that the revenue feature of the law appeared to be all important, and that if it was the purpose of the legislature to exercise only the police power it had gone too far in adopting such a high scale of fees, the court ruled, therefore, that the statute was void in that it violated the due process clause of the State and Federal Constitutions.

California in 1913 passed a motor-vehicle act which provided for a scale of license fees based on horsepower. The law was contested in the courts and its constitutionality was passed upon by the supreme court in 1914. It was contended by those opposing the law that the classification of motor vehicles according to horsepower was illegal, but the court in upholding the law stated that it regarded this method of classification as "an attempt on the part of the legislature to apportion the license fees with some reference to the destructive or wearing power of each motor vehicle." (Ex parte Schuler, 167 Cal. 282.) While the horsepower basis might not be considered as an absolutely scientific method of determining road wear, the court could not say that it was an unjustified method. Automobiles of high power were usually heavier than automobiles of low power and the damage done to the road by the heavier vehicles was considered greater than that done by the lighter cars. Speed, which is another factor in road wear, was not mentioned by the court.

UNITED STATES SUPREME COURT DECLARES LAWS CONSTITUTIONAL

The constitutionality of legislation passed by States providing for a license tax upon automobiles to be paid by both residents and nonresidents was finally passed upon by the United States Supreme Court. The New Jersey case was appealed to the Federal courts and another case arose out of a similar enactment by Maryland. (Hendrick *v.* Maryland, 235 U. S. 610 (1915) Kane *v.* New Jersey, 242 U. S. 160 (1916). In unanimous opinions the Supreme Court held that the licensing of automobiles and of drivers and the charging of fees graduated according to horsepower was an exercise of the police power of a State and that its purpose was to preserve the health, safety, and comfort of its citizens. The revenue features of this character of legislation were also upheld by the Supreme Court on the grounds that when a State at its own expense furnished special facilities, such as improved highways, it might charge special compensation for their use whether the users were residents or nonresidents. After these decisions there was no longer any doubt as to the right of the States to charge motor vehicle license fees which were designed to produce highway revenues in addition to the amounts of money necessary to administer such licensing laws as a part of a State's police power. The fees paid could not be regarded as property taxes and the States were authorized to establish reasonable classifications upon which such charges could be based.

LITIGATION WITH RESPECT TO GASOLINE TAXES

When a number of the States began to pass laws providing for a tax upon gasoline a new phase of motor-vehicle taxation was developed. The courts were again invoked to determine whether or not such a tax could be considered valid upon constitutional grounds. The Supreme Court of Arkansas was called upon to review this question soon after the passage of an act of the legislature (1921) instituting a tax of 1 cent per gallon upon all gasoline sold within the State which was to be used in propelling motor vehicles. The law provided that all persons or corporations selling gasoline to be used in propelling motor vehicles over the highways of the State should collect from the purchaser of such gasoline 1 cent per gallon in addition to the regular price and remit the amount of money thus collected to the State, which in turn would distribute one-half to the general road fund of the county in which the tax was collected and one-half to the State highway improvement fund. Other sections of the act required wholesale distributors and retail dealers to file monthly reports showing the sales of gasoline used for such purposes.

The opponents of this new kind of tax contended that it constituted a property tax on gasoline and was therefore void because it violated the constitutional provision of uniformity. The question for the court to decide was whether or not the gasoline tax was a property tax, because it was conceded by all parties to the action that if it was there was a violation of the uniformity principle. (Standard Oil Co. *v.* Brodie, 239 S. W. 753 (Ark.).) The language of the act disclosed that it was neither the intention of the legislature to impose a tax upon gasoline nor upon its sale, nor even upon its use, but that the tax was essentially a tax upon the use of motor vehicles on the public highways. The extent of the use was measured by the amount of gasoline consumed which resulted in a tax on motor-vehicle operation in accordance with the use made of the highways. This interpretation disposed of the objection to the tax on the grounds that it was a property tax.

It was also argued by the objectors to this statute that it was a species of double taxation in that the legislature had already placed a tax upon the users of the highway in the form of an automobile license fee. The court disposed of this proposition in a rather ingenious fashion. It held that the license fee could be regarded as a tax upon the privilege of using the highways according to the capacity of the car, while the gasoline tax was an additional tax upon the privilege of using the highways according to the extent of the actual use. In adopting this point of view the court adopted the principle of charging for highway use which has been developed in the theory of public-utility rates and charges. Electric-power rates, for example, are based upon these same two factors—capacity and use; a flat readiness-to-serve charge is made, based on the consumer's capacity to use the service, and an additional charge is made for the current actually used. The former charge is designed to cover to a large extent the capacity costs or fixed charges and the latter charge covers the direct operating expenses. The gasoline tax in combination with an annual license fee follows the same principle.

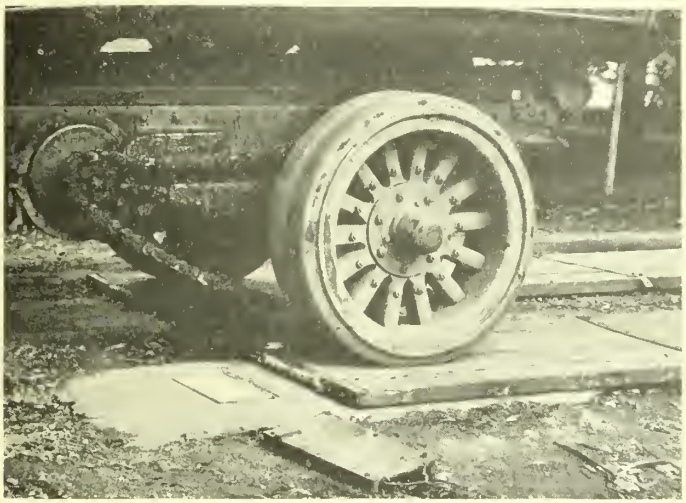
It was also brought to the attention of the court that the gasoline tax afforded means of evasion to those motor-vehicle operators who lived close to the borders of the State and who could purchase their supplies of gasoline in adjoining States which did not have a gasoline tax and that such a possibility of evasion constituted a discrimination against others who were compelled by their location to pay the tax. The court admitted the possibility of such an evasion of the gasoline tax, but held that it did not render the statute arbitrarily discriminatory in a legal sense.

(Continued on page 14.)

STATUS OF THE MOTOR TRUCK IMPACT TESTS OF THE BUREAU OF PUBLIC ROADS

By C. A. HOGENTOGLER, Associate Highway Engineer, United States Bureau of Public Roads

SIX years ago the Bureau of Public Roads undertook an investigation of motor-truck impact. The object of the research was two-fold—first, to determine the forces to which pavements are subjected, and, second, to determine the resistance to these forces of various types of road surfaces.



A truck wheel about to drop from a 2-inch elevation upon the piston head of the hydraulic jack used for transmission of the impact to the copper cylinders. The channel guard in the lower foreground was used to prevent recording the impact of the front wheels. It was pulled away in time for the passage of the rear wheels

From time to time reports of several phases of the investigation have been published in PUBLIC ROADS,[1] all of which have dealt with the measurement of impact forces and the effect of such forces on specimen slabs of pavement constructed at the Arlington Experimental Farm, Arlington, Va.

In the first stages of the investigation efforts were directed to the determination of the forces of impact delivered by moving trucks to road surfaces. The effect of the impact upon the surface was not considered. To determine the impact forces, motor trucks were driven over obstructions or drops placed in the roadway and the impact of the wheels on striking the obstruction or falling over the drop was received on the piston head of a hydraulic jack which transmitted the force of the blow to a small copper cylinder set between the piston and the bottom of the jack cylinder. The copper cylinders were calibrated under static pressures and their deformation by any given static force was known. It was assumed that an impact which caused deformation of the copper cylinders equal to that caused by a given static force would be equal to the static force. While these tests indicated that the impact of a motor truck wheel striking the surface of a road might be several times as great as the static load on the truck wheel, it was found that the impact force indicated was not as great as the maximum impact which would be delivered to the road without interposition of the copper cylinder, for two reasons: First, the copper cylinder itself, in deforming under the impact, to that extent cushioned the blow and, by reducing the deceleration, reduced the impact; second, the copper cylinder deformation was a measure of the work done upon the cylinder and not the maximum impact delivered to it. At the instant of contact of wheel and piston head the force of impact is zero. As deformation of the copper cylinder progresses the impact increases. The average force or resistance of the copper multiplied by the deformation or distance through which it acts is equal to the energy applied (heating and elastic effect neglected). This reasoning led to the conclusion that the force measured by the copper cylinder was the average force of impact and not the maximum.

THE TESTING OF SLABS BEGUN

The next phase of the investigation introduced the testing of sections or slabs of pavement of various kinds to ascertain their behavior under impact. To deliver the impact a machine was designed which, as shown by the illustration on page 12, consisted of a loaded box riding on a 5½-ton truck spring which, in turn, was supported by a loaded plunger on the bottom of which was a double 2-inch by 6-inch solid rubber tire. The box and plunger representing the sprung and unsprung weights, respectively, could be so loaded as to represent a truck of any size. By means of a motor, gears, and cam the plunger or unpsrung weight carrying the spring and sprung weight could be lifted and dropped from any height so that the effect would be identical with that of a truck wheel dropping from one level to another.

Looking down on the hydraulic jack used for transmission of impact to copper cylinders. The copper cylinder held in the hand is placed in the bottom of the jack cylinder. The blow of the truck wheel is delivered to it through the piston

[1] "The how and why of truck impact," Public Roads, volume 3, No. 31, November, 1920.
"Motor truck impact tests of the Bureau of Public Roads," Public Roads, volume 3, No. 35, March, 1921.
"Tests of impact on pavements by the Bureau of Public Roads," Public Roads volume 4, No. 6, October, 1921.
"Tests of impact on pavements by the Bureau of Public Roads," Public Roads, volume 4, No. 7, November, 1921.
"Motor truck impact tests of the Bureau of Public Roads," Public Roads, volume 4, No. 8, November, 1921.
"Impact tests on concrete pavement slabs," Public Roads, volume 5, No. 2, April, 1924.

Impacts delivered in this way were applied to a series of 7-foot square slabs of various paving materials. The problem of measuring the force of the blows delivered was met by the development of an autographic process for determining the maximum declaration of the falling load. Multiplying this deceleration by the mass of the

A general view of the first type of impact machine

plunger, spring, and loads gave as a result the impact in pounds corresponding to any drop of the machine. The autographic apparatus consisted of a strip of silicated paper which was moved horizontally over a 5-inch drum at a rate recorded by means of an instrument designed to break electrical contact every second and thus cause a movement of a time-recording point against the paper. Three other points attached, respectively, to the unsprung weight, the sprung weight, and the slab and bearing against the moving paper recorded the vertical movements of each when the plunger was dropped upon the slab. The graphs thus formed measured time by their horizontal distances and vertical movements by vertical distances. By mathematical analysis of them it was possible to derive the velocity of the falling load and the deceleration of the velocity from the instant when the rubber tire struck the slab until the downward velocity was reduced to zero. For the purpose of such analysis the autographic curves, a typical sample of which is reproduced on this page, were enlarged several times by means of an accurate pantograph.

Apparatus for recording space-time curves of motor truck impact

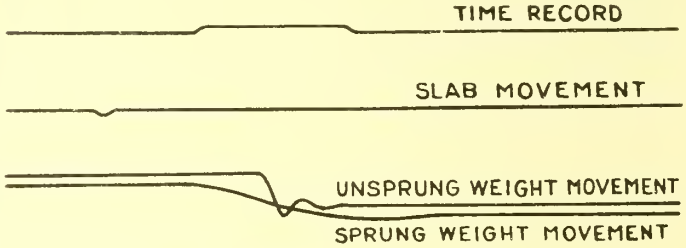
Impact forces delivered to the first series of experimental slabs were measured in this way and their effect on the slabs was determined by noting the height of fall and impact required to cause failure of the various slabs. Reports of this series of tests were published in PUBLIC ROADS for October, November, and December, 1921.

THE TESTING OF THE SECOND SERIES OF SLABS

Upon the completion of these tests a new series of slabs was constructed including a wider range of designs, and work was begun on the construction of a new type of machine for delivering and measuring impact forces. The machine which was finally developed was far more compact and easier to move from slab to slab than the one used in the earlier tests; and for the measurement of the impact force the autographic space-time apparatus was supplanted by an accelerometer to measure the acceleration directly and reduce the time and labor required in the interpretation of the test results.

The work of developing and calibrating a type of accelerometer suitable for the purpose was one of the most difficult phases of the entire investigation. The problems involved were satisfactorily solved, however, and the perfected accelerometer was used in connection with the new impact machine for the testing of the second series of slabs, the results of which were published in PUBLIC ROADS for April, 1924.

In the testing of the first series of slabs the behavior of the slabs could be determined only by observation of the occurrence of cracks and the deflection of the slabs following the blows delivered. No means were available for the measurement of the deformation of the slabs under the impact blows. Therefore the development of the graphic strain gauge which, when embedded



A typical space-time record of a single drop of the impact machine

in the slab, was found to give an accurate record of the deformation of the experimental section constituted one of the distinct advances of the second series of slab experiments.

In all the work up to this time the actual impact delivered by motor trucks to road surfaces was measured at one location on an actual road and corresponding impacts were then delivered to the experimental pavement slabs by means of the impact machines at another location.

THE CURRENT INVESTIGATIONS

The investigation has now entered a third stage in which the force of the impact delivered by an actual truck to an actual road is measured simultaneously with the effect of the blow on the road surface. Records of accelerometers mounted on the moving truck permit computations for the former, while graphic strain gauges inserted in the road surface indicate the stress produced in the slab, and thus show the latter.

In determining the magnitude of impacts found on ordinary roads, the current investigation goes further than those that preceded it in that it inquires into the causes of the impacts as well as their magnitudes. In this study the profile of the road surface as given by a 16-wheel profilometer is correlated with accelerometer records. This phase of the work in itself should be of special interest to the construction engineer since it opens up a means of determining whether or not a finished pavement has the proper degree of

smoothness. If it should not have, records from these instruments will indicate the exact location of the troublesome areas or obnoxious bumps. It is not an extravagant prediction to say that in the very near future the acceptable degree of road smoothness will be determined by properly standardized instruments either mounted on a standard vehicle or towed behind it at specified speeds. The State highway departments of both New York and Illinois are now experimenting with apparatus of this character. New York uses the Vialog, an instrument which is attached to an automobile and which measures the deflection of the front spring, while in Illinois a 16-wheel profilometer is drawn by a truck and gives an accurate profile of the road.

Briefly stated, the present investigation was designed to secure definite information in regard to the wheel impacts as influenced by road roughness, vehicle design, load, speed, and tire equipment, and, as stated before, the resistance of the various pavement surfaces to these impacts.

PROCEDURE IN THE CURRENT INVESTIGATIONS

Procedure in this investigation is simple. First, representative sections of the various types of pavements commonly used, such as concrete, brick, granite block, and bituminous combinations are selected and marked off. A general indication of the roughness of these sections as compared with that of all available lengths of similar pavement is secured from an instrument similar to the Vialog which is mounted on a touring car. The actual profiles of the test sections are secured by means of a 16-wheel profilometer.

Over these selected sections of road are then run the test trucks, which range from 1 to 5 tons capacity. The truck variables include four speeds and three loadings—empty, capacity, and overload—and four kinds of tires—pneumatic, cushion, new and worn solid. By means of accelerometers attached to the trucks, records of the accelerations and the accompanying spring deflections and speed of travel will be obtained. With this information and a knowledge of the sprung and unsprung weights, a computation of the force of the blow is quickly made.

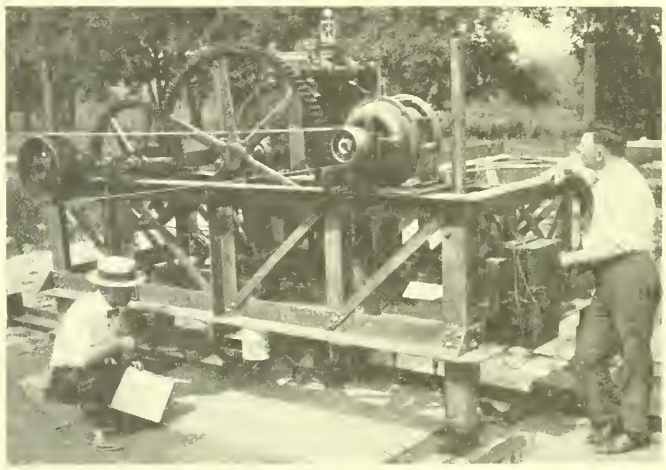
After the impacts occurring on normal roads have been determined, they will be reproduced on the experimental test road and their effects will be recorded by the graphic strain gauges which indicate the stress occurring in the concrete sections. This procedure has been made possible only because of the development of the graphic strain gauge and the accelerometers.

The development and calibration of the apparatus constitute the most difficult phase of the impact investigations. In the case of the graphic strain gauge it was essential to have a device which would accurately measure fiber deformations ranging from zero to several ten-thousandths of an inch and at the same time be so small as to be conveniently inserted in a road slab under a truck wheel. While several accelerometers are available commercially, none of them answered the test requirements, which were that the apparatus must be capable of measuring accelerations ranging from zero to 1,000 feet per second and at the same time be sturdy enough to function properly when subjected to the vibration caused by its being attached to a rapidly moving motor truck. In addition to the mechanical difficulties, we were faced by the fact that authorities disagreed as to the practicability of securing usable force values from accelerometer records.

THE ACCURACY OF THE ACCELEROMETER RECORDS

The feeling of doubt existing among engineers as to the practicability of determining impact forces from accelerometer records is expressed by C. D. Young, past president of the American Society for Testing Materials, in a discussion of a paper entitled "Accelerometer for measuring impact," presented before that body by E. B. Smith, of the Bureau of Public Roads Mr. Young is quoted as follows:

> The earliest testing of railway materials was largely by the impact method, namely, by drop testing. That is still the case in the testing of rails. At one time couplers, bolts, and a great deal of railroad material was tested by impact. Efforts were made to translate the results of those impact tests into force in pounds. I believe it was the consensus of opinion through 20 or 30 years' experience in that work that it was neither wise nor safe to draw too definite conclusions in the translation of the forces applied through acceleration into pounds loading equivalent to static. A great deal depends upon the character of the apparatus as to what the ratio factor should be in that translation. A great deal depends upon the mass back of the resistance, and, as I understand the paper, in this particular case it would be the mass of the vehicle carrying the instrument. In addition to that the character of the springs would be a determining factor in the translation of those forces into pounds, in the same way that the character of the spring bed supporting the anvil of a



The latest type of impact machine

> drop-testing machine has its effect upon the translation of the falling tup to the effect upon the metal and deflection of the metal. In other words, by changing springs in the anvil or the support of the piece deflected under accelerated drop test you can decidedly change the deflection for the same height of fall with the same material. So I feel that where data are obtained through accelerated movement, as in this instance, the attempt to translate into pounds of force should be made very cautiously.[2]

Happily, it can now be said that the calibration of the instruments has cleared away the doubts and has substantiated previous assumptions and shown conclusively that the instruments to be used in the current investigation accurately perform the work for which they were designed.

When all of the data from this investigation have finally been secured and analyzed, it is confidently hoped that it will be possible to say definitely just what types of vehicles, tires, and loads the various road surfaces can safely support. By correlating this information with that secured from traffic surveys, a simple mathematical calculation will suffice to determine the best combinations of road and vehicle for economic motor transport. Information of this character is of prime importance to the legislator as well as the engineer.

[2] Proceedings of the American Society for Testing Materials, vol. 23, 1923, Pt. II p. 633.

LEGISLATION SHOULD WAIT FOR TEST RESULTS

With the rapid development of motor vehicle transportation need for new laws and regulations arose, and in the absence of definite information many of them were based on more or less scientific guesses. For instance, many laws are based on total wheel load alone. They serve their purpose very well when the truck is not in motion, for a wheel at rest is a definite, tangible quantity which can exert no greater pressure than its static weight. The instant the wheel is set in motion, however, a different condition develops.

As the truck moves along a road surface variations, tire roughness, variations in rubber composition, etc., cause the wheels to mount vertically, after which the force of gravity and spring pressure cause them to return to the road, and make them capable of delivering forces ranging from their static weights to amounts eight or ten times as great. These forces are dependent upon the vertical velocities acquired and the time or distance in which these velocities are reduced to zero. That this change in velocity or deceleration as well as the accompanying force is due largely to the type of cushion existing between the falling body and stopping agency is shown by the following experiment:

A lead weigh of 9 pounds was dropped from the same height—one-half-inch—on five different springs and the resulting forces indicated by the spring deflections were recorded. These springs were previously calibrated under static loads and were found to have stiffness ranging from 31 to 400 pounds per inch deflection. The 9-pound weight when dropped on the 31-pound spring indicated a pressure of 50 pounds, when dropped on the 50-pound spring a pressure of 68 pounds was registered, on the 190-pound spring the force of the blow was 121 pounds, and when dropped on the 400-pound spring, a pressure of 173 pounds was recorded.

It will be remembered that the forces in the first impact tests were computed from a relation between the falling weight, the total fall, and the cushion or the distance in which the weight was brought to rest. Applying that same relation to a considerable number of drops from different heights of the 9-pound weight on the five different springs, a curve was secured which indicates that so long as an elastic cushion is used the maximum force of the blow is entirely dependent upon the percentage of cushion to total height of fall. This relation held with few exceptions, for the impacts delivered by the truck impact machine during the recent calibration work just as it did for the small springs.

Data available to date indicate that pressures delivered by the wheels of a moving motor truck vary just as much as those of the small weight cited above. Our previous tests have shown that a badly worn solid tire can deliver an impact seven times as great as the static wheel load. In one instance the difference in corner deflection caused by a 2,000-pound wheel load with new and very badly worn solid tires was 1,600 per cent. It is interesting to note in this case that the greatest deflection under the 2,000-pound wheel load was double the normal deflection under an 8,000-pound wheel load with a new solid tire. Pneumatic tires, on the other hand, seem to definitely limit the impact. In no case, using obstructions up to and as high as 4 inches, has it been possible to record pressures under pneumatic tires greater than double the static weight. Vehicle design also causes variation in wheel pressure. During the present tests we have indications that with one type of truck accelerations increase with both height of drop and speed, while with another a limiting impact seems to have been reached before maximum drop and speed have been reached.

When, as noted above, it is indicated that a 2,000-pound load can be more detrimental to a slab than one of 8,000 pounds, the fallacy of basing design and legislation on the wheel load is plainly seen. The above results, however, can be taken only as indications, since they are the result in some cases of questionable assumptions. We are now equipped with instruments and knowledge with which all of these questions can be quickly and definitely settled. That is the status of the impact tests at present. The most difficult part of the work, that of securing proper instruments of measurement and determining correct methods of interpreting results, has been completed. From this time it is expected that results from actual road tests will be secured rapidly.

(Continued from page 10)

UNITED STATES SUPREME COURT PASSES ON VALIDITY OF GASOLINE TAXES

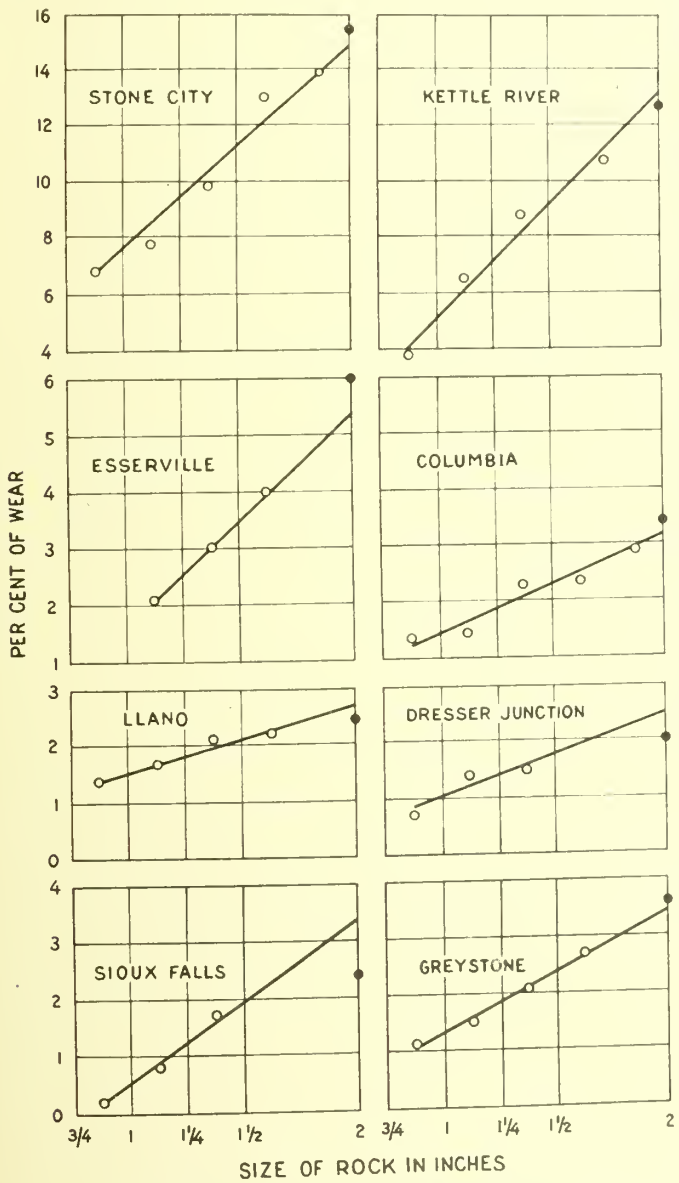
Although the State supreme court upheld the constitutionality of the gasoline tax statute passed by the Arkansas legislature, efforts were made by one of the large oil companies to have the law set aside as being in conflict with the Federal Constitution, and proceedings were brought in the district Federal court to enjoin the tax officials from the collection of the tax. The matter was finally appealed to the United States Supreme Court, which held that it did not come into conflict with the Federal Constitution. (Pierce Oil Corporation *v.* Luther Hopkins et al. (Decided Feb. 18, 1924).) It was claimed that the due-process clause of the Constitution was violated in that the tax was levied as a privilege tax for the use of the highways upon the purchasers of the gasoline but that the collection of the tax was imposed upon the distributors, who were obliged to make monthly payments and reports, and that this constituted an appreciable expense to them. The court refused to consider this a denial of any constitutional rights, and in doing so stated that a State had the power to regulate the business of selling gasoline and also the power to tax the privilege of carrying on that business and that the due process clause of the Federal Constitution did not prevent the State from imposing this incidental burden growing out of the collection of a gasoline tax.

This analysis of State and Federal court opinions indicates that the principle of licensing motor vehicles both for the purpose of police regulation and for highway revenues has been definitely approved by the courts as to its constitutionality. The gasoline tax likewise has been found to be not in conflict with any constitutional provisions except in so far as certain States have constitutions which definitely prohibit revenue measures of that kind. Where such situations exist they can and are being met by amendments submitted to the voters for their approval in cases where State legislatures contemplate the introduction of a gasoline tax to secure revenues for highway purposes.

ROAD MATERIAL TESTS AND INSPECTION NEWS

AN ABRASION TEST FOR CRUSHED STONE

THE Bureau of Public Roads wishes to call attention to a test for abrasion which may be applied to samples of crushed stone. As is well known, the present Deval abrasion test is only adapted to the testing of rock samples where the individual fragments are at least 2 inches in diameter. No method has as yet been adopted for testing crushed stone in the smaller sizes, such as ¾ inch, 1 inch or 1½ inches. The bureau has been endeavoring to develop a test of this nature for some time. Although the results so far secured are not sufficiently complete to warrant a recommendation for its general adoption it is believed that this method, which has so far given promising results, should be described for the benefit of others who may desire to work along the same line.



Abrasion loss by crushed stone particles of various sizes

The method of test is as follows: The sample as received is separated by means of screens with round openings into as many of the following sizes as its gradation will permit: ¾ to 1 inch, 1 to 1¼ inches, 1¼ to 1½ inches, 1½ to 2 inches. Samples for test are then secured by selecting 50 pieces of each size, which are tested for abrasion separately in accordance with the usual procedure. When a sample of crushed stone is tested in this manner and the abrasion losses compared as in the accompanying chart, an interesting relation is observed, which may be stated as follows: For a given sample of crushed stone the loss by abrasion for each size tested is proportional to the size of the pieces composing the test charge, provided that the same number of pieces are used in each case. This relationship is, of course, only approximately true. There are many factors which tend to impair it, the most important of which are variations in shape of the fragments, slight variations in size, etc. For this reason it is well whenever making tests of this nature to secure at least duplicate results for each size.

Assuming, however, that the relationship is approximately true, this method of test presents a means of estimating the probable percentage of wear by the standard method in cases where it is impossible to make this determination directly. Referring to the accompanying graph, it will be noted that the straight line averaging the various points for each sample is extended to the right until it intersects the ordinate for the 2-inch size. The solid circles on the 2-inch ordinates in each chart represent the percentage of loss by the standard Deval abrasion test. It will be observed that in only one of the eight cases plotted does the point where the straight line intersects the 2-inch ordinate differ more than 0.6 per cent from the standard test result. This is true in spite of the fact that the standard test results were secured upon hand-broken samples, whereas the other values were obtained by testing the product of the crusher. The discrepancies that appear are caused without doubt by this difference in the character of the test pieces.

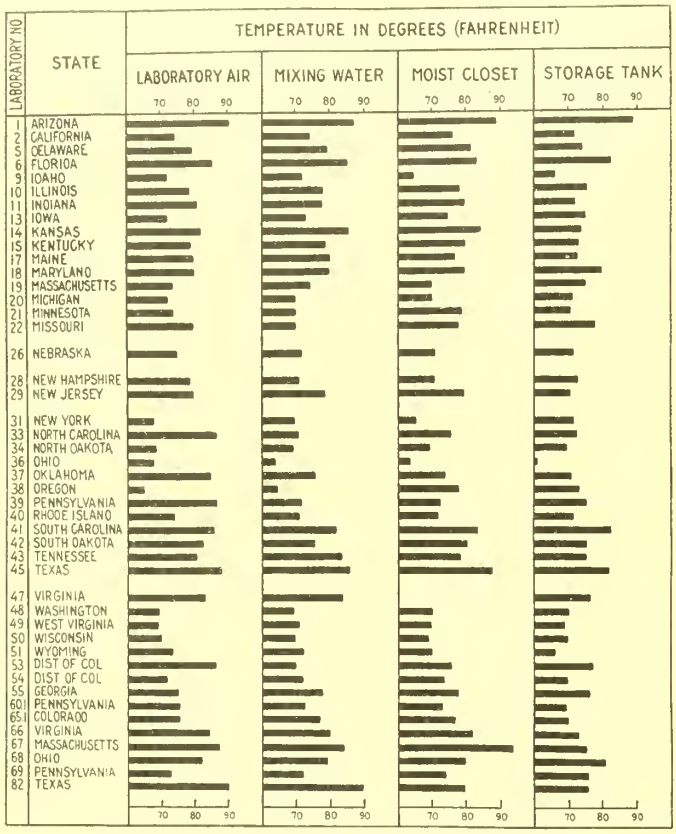
CHECK TESTS OF MATERIALS

The system of check testing materials undertaken by the Bureau of Public Roads, in cooperation with laboratories located in various parts of the country, is continually yielding information of great interest to testing engineers.

It has been possible already to detect and correct many irregularities in laboratory procedure, most of which were found to be due either to the lack of standard equipment or to the failure of operators to appreciate the importance of details. The bureau desires to take this opportunity of thanking the various laboratories for their cooperation in the conduct of this work.

In connection with the results of check tests of cement, Ct–5, it seems advisable to call attention at this

time to a condition in regard to temperatures which appears to be quite common. The accompanying chart gives the temperatures of the laboratory air, mixing water, moist closet, and storage water which were reported by the 60 laboratories cooperating in this test. It will be observed that there is a general tendency to work at temperatures considerably above the standard of 70° F. set by the specifications. These tests were made during the latter part of July and the 1st of August, 1924, and the values are, of course, higher than they would be in the winter throughout the Northern States. The great bulk of cement testing, however, is done during the spring and summer months when temperatures are likely to be higher everywhere. Although the exact amount of variation from standard results that may be due to variations in temperature alone is open to question, there can be no doubt that efforts should be made to secure closer adherence to standard requirements than is here revealed. This is particularly true of both the mixing water and moist closet temperatures, both of which quite appreciably affect the test results.



Temperatures reported in check tests of cement sample Ct-5

In their inspection of laboratory conditions throughout the country, representatives of the Bureau of Public Roads have found in general that very little attempt is made to control this variable in the cement laboratory. This is due in almost all cases to the expense involved in the installation of the proper form of automatic temperature control apparatus. But, when one considers that the disposition of cement worth from $400 to $500 depends upon the result of a single test it appears highly desirable that every precaution possible be taken to insure compliance with every detail of the specification.

It is unfortunate that most of the apparatus which is adapted for controlling temperatures automatically is expensive. This is especially true of any automatic scheme for lowering temperatures. A complete refrigeration system is needed for best results, although conditions can be somewhat improved by the use of ice. Apparatus for raising temperatures to the standard, on the other hand, is quite reasonable in price and easy to install. For this reason it might be well to give consideration to the proposition of raising the standard temperature from 70° F. (21° C.) to 77° F. (25° C.) which is the standard for most laboratory operations. It is believed that compliance with the temperature requirements of the specification could be much more easily secured if this were done.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publcation to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

DEPARTMENT BULLETINS

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*136. Highway Bonds. 20c.
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*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10e.
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No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Seeretary of Agriculture for use in connection with Federal aid road construction.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
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*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15e.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
74. State Highway Mileage and Expenditures for the Calendar Year 1916.
161. Rules and Regulations of the Secretary of Agrieulture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D–2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D–4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D–6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D–13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D–15. Tests of a Large-Sized Reinforeed-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

OCTOBER 31,1924

Government Printing Office

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,185,247.54	464.1	$ 575,007.24	$ 284,651.85	85.1	$16,530,308.90	$ 7,634,554.61	835.6	$ 79,415.15	$ 39,708.07	0.7	$ 1,207,900.32	Alabama
Arizona	8,338,365.41	4,287,583.98	627.8	713,173.97	406,926.70	37.5	1,730,194.41	1,048,007.14	150.4	387,292.14	235,674.21	40.7	1,516,510.07	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	1,017,822.49	433,389.63	47.7	7,081,513.93	2,567,091.62	355.4	499,080.76	216,952.35	47.1	1,420,620.77	Arkansas
California	12,999,075.03	5,647,148.17	533.7	3,169,801.35	1,597,213.27	122.4	12,310,508.18	5,401,783.18	407.2	435,211.04	255,774.51	1.3	3,090,386.87	California
Colorado	8,108,070.31	4,029,898.97	502.6	724,284.07	374,740.49	39.3	5,385,426.62	2,920,403.94	204.6	62,567.99	35,168.34	5.1	2,199,669.26	Colorado
Connecticut	3,062,872.02	1,259,559.60	73.6	199,024.56	98,423.00	4.9	3,252,054.48	1,045,804.04	54.1				967,409.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	462,969.09	197,825.82	13.9	1,196,267.49	449,291.10	29.4	122,520.75	55,950.00	3.7	29,768.26	Delaware
Florida	961,134.07	461,470.92	49.8	903,102.67	429,096.42	13.0	8,954,456.15	4,312,794.00	248.8	398,642.71	171,123.94	10.3	912,411.72	Florida
Georgia	17,167,373.32	7,955,805.20	1,214.2	719,529.54	357,915.42	51.1	10,970,229.25	5,404,189.33	823.9	935,377.74	372,029.44	31.3	359,957.61	Georgia
Idaho	8,181,697.92	4,092,395.62	505.8	243,857.57	139,952.52	18.7	2,491,399.16	1,496,692.99	170.5	63,387.46	36,764.71	5.0	911,906.26	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	1,355,789.15	570,702.08	40.5	15,741,331.50	7,820,073.10	538.8				2,656,170.49	Illinois
Indiana	7,577,444.16	3,655,540.97	225.7	1,202,211.53	590,703.25	41.5	15,162,550.11	7,375,195.57	517.4				2,190,952.21	Indiana
Iowa	23,195,778.19	9,237,031.86	1,682.9	282,803.25	113,620.08	20.2	8,764,543.61	4,071,449.47	637.7	1,125,849.69	505,300.00	62.4	1,408,735.69	Iowa
Kansas	17,094,135.48	6,043,176.90	502.7	4,996,700.82	1,901,478.91	146.0	14,890,305.54	5,952,371.64	516.5	1,900,933.68	877,366.90	108.7	514,994.95	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	845,043.61	359,276.29	30.2	9,053,977.00	4,197,562.66	355.0	474,276.14	204,477.96	17.0	1,006,474.92	Kentucky
Louisiana	8,488,463.18	3,636,143.36	561.2	1,149,422.02	559,243.59	104.9	6,017,449.19	2,493,750.83	306.1	191,242.29	95,621.14	1.1	480,683.08	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	295,533.42	147,087.77	12.3	1,704,509.38	823,525.56	62.5	113,567.57	56,833.78	5.3	762,589.51	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	368,411.02	179,205.49	16.4	2,555,937.34	1,097,755.29	77.8	414,509.32	154,670.00	11.6	13,997.44	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	88,434.91	28,750.00	1.4	6,480,878.47	2,091,214.34	106.9	466,585.01	137,365.00	7.2	1,556,713.44	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5	85,790.03	41,110.20	7.3	15,725,921.72	7,492,390.61	534.4	227,396.58	89,025.00	6.0	2,195,633.96	Michigan
Minnesota	24,037,561.24	9,885,843.07	2,298.0	1,390,461.31	549,367.01	107.2	9,973,211.56	4,562,729.97	862.7	682,952.36	109,300.00	74.8	111,188.95	Minnesota
Mississippi	7,888,193.99	3,828,941.39	555.0	727,671.41	363,202.88	50.7	8,249,905.07	4,110,761.62	500.2	334,543.17	167,274.07	17.8	1,061,093.14	Mississippi
Missouri	11,352,027.70	5,245,999.18	803.5	1,317,280.96	550,328.61	100.6	19,419,957.57	9,240,113.31	794.3	2,110,864.83	1,031,504.24	125.2	1,772,342.66	Missouri
Montana	8,857,279.16	4,384,335.12	791.4	553,942.63	425,622.05	77.2	2,315,900.94	1,755,169.14	207.3	1,068,684.29	606,876.40	119.5	3,793,414.29	Montana
Nebraska	7,875,337.16	3,714,691.59	1,440.4	367,154.70	170,993.65	26.9	7,804,592.90	3,919,775.94	823.8	859,633.73	429,766.82	87.5	3,315,718.00	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.5	500,051.46	522,259.72	43.4	4,919,310.57	4,122,474.82	423.5	89,227.45	77,824.18	8.5	314,128.30	Nevada
New Hampshire	3,075,750.19	1,487,867.58	171.3	39,659.20	19,277.89	1.9	1,592,552.32	798,211.95	56.5	23,510.71	11,755.35	0.3	117,851.23	New Hampshire
New Jersey	7,623,795.12	2,561,531.49	149.7	298,751.09	111,360.00	6.5	10,521,979.85	3,001,281.24	84.1				915,074.27	New Jersey
New Mexico	5,306,286.45	2,758,849.58	714.3	459,922.73	279,146.53	59.5	5,713,439.94	4,403,518.25	651.8	250,983.39	154,354.77	16.5	993,462.77	New Mexico
New York	18,962,742.49	8,257,844.44	572.7	501,393.18	258,083.36	19.2	30,094,966.96	11,306,316.72	668.5	6,378,094.00	1,790,545.00	115.9	5,095,358.48	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	1,076,292.98	422,979.39	21.3	10,700,920.49	4,039,779.09	312.1	1,822,902.70	863,142.71	48.8	1,295,592.15	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1,587.9	891,995.28	445,399.39	184.8	3,184,046.35	1,579,002.17	473.1	425,922.84	213,411.38	36.2	1,707,337.64	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	1,772,058.39	693,050.64	44.6	13,962,924.71	5,175,505.01	382.9	914,126.13	363,245.62	37.6	2,028,442.74	Ohio
Oklahoma	12,985,865.26	5,989,852.03	497.3	1,193,003.76	575,235.49	62.9	8,943,018.46	4,280,990.91	380.1	1,735,045.71	755,679.93	89.4	1,036,944.64	Oklahoma
Oregon	12,082,873.17	5,819,093.79	555.5	808,472.49	469,237.85	89.6	3,127,155.01	1,845,453.31	167.6				373,374.05	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	738,455.97	318,587.37	16.8	22,994,562.90	6,297,181.25	382.9	2,137,715.79	609,581.25	39.9	3,261,371.34	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	45.0	486,927.24	210,838.14	11.2	869,747.59	285,169.30	17.1	380,178.15	92,700.00	6.2	564,105.50	Rhode Island
South Carolina	9,015,476.73	4,124,045.22	924.4	1,118,224.21	497,540.74	169.2	5,691,011.81	2,356,889.68	410.8	519,424.14	137,323.95	45.8	671,745.50	South Carolina
South Dakota	8,674,597.85	4,244,636.27	989.8	845,706.40	453,412.58	120.0	8,026,510.71	3,975,940.91	1084.6	87,989.45	40,895.15	14.3	3,795.09	South Dakota
Tennessee	6,805,583.35	3,313,935.07	259.6	2,919,375.46	1,434,944.87	101.3	12,933,199.49	5,798,648.30	411.4	1,173,785.49	577,049.19	63.6	900,058.57	Tennessee
Texas	42,341,998.56	16,190,624.91	3122.8	3,154,465.53	1,395,235.58	213.6	25,104,633.25	9,934,262.23	1572.9	3,008,806.31	1,170,852.25	141.8	3,033,238.02	Texas
Utah	3,304,423.75	1,895,805.92	219.0	719,066.79	390,456.24	91.2	4,591,270.78	3,136,607.08	278.0	179,027.65	114,931.00	13.0	578,672.76	Utah
Vermont	1,922,114.16	942,759.12	74.4				2,119,474.83	1,013,156.18	53.4	175,767.00	79,026.96	5.1	499,026.74	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	412,925.70	165,709.43	11.6	10,397,592.18	4,772,627.88	362.4	121,041.73	49,466.96	3.1	803,366.30	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	232,694.93	109,880.09	23.4	4,172,115.91	1,912,300.00	153.8	76,148.13	38,000.00	5.2	535,602.46	Washington
West Virginia	5,489,747.95	2,365,041.63	255.6	1,030,538.45	465,652.95	31.3	4,786,142.71	2,150,604.61	185.5	44,213.40	16,800.00	1.1	756,022.91	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1,325.3	642,850.91	310,652.72	33.3	4,587,966.47	2,208,743.02	189.5	195,697.26	93,744.00	8.4	3,624,277.69	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	811,510.31	471,419.85	92.7	4,405,501.89	2,784,637.92	307.1				353,194.53	Wyoming
Hawaii													365,625.00	Hawaii
TOTALS	$549,656,391.27	$237,862,399.82	32452.9	$44,709,573.68	$21,291,205.90	2643.1	$417,204,570.43	$187,753,733.63	19227.7	$ 32,695,037.94	$13,132,967.35	1481.0	$ 65,094,703.30	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 84,083,419.11 Federal aid $ 37,909,961.69 Miles 3,689.4

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 10 ▽ DECEMBER, 1924

FOR MANY YEARS THE WHEEL SCRAPER HAS BEEN A STANDARD GRADING IMPLEMENT

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U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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ACCURATE ACCELEROMETERS DEVELOPED BY THE BUREAU OF PUBLIC ROADS

By Leslie W. Teller, Assistant Testing Engineer, United States Bureau of Public Roads

IN the review of the progress of the impact tests of the Bureau of Public Roads, published in the preceding number of this magazine,[1] the various methods which have been employed in the attempt to measure the force of the impact delivered to a road surface by the wheels of motor trucks were briefly described, and some of the problems and difficulties attending these attempts were indicated. The article recalled the fact that the first measurements were made with calibrated copper cylinders the deformation of which under impact was used as the measure of the force of the blow. The cylinders having been previously calibrated with respect to their deformation under known static loads, the impact forces which produced equal deformations were assumed to be equivalent to the corresponding static loads. The error of this assumption was pointed out and the article described the course of the subsequent experiments which have led to the perfection of a method of determining the impact forces by the measurement of the deceleration of the vertical motion of the truck wheel when it is arrested by the resistance of the pavement. This method is an application of the well-known principle that the force of impact is equal to the product of mass times acceleration.

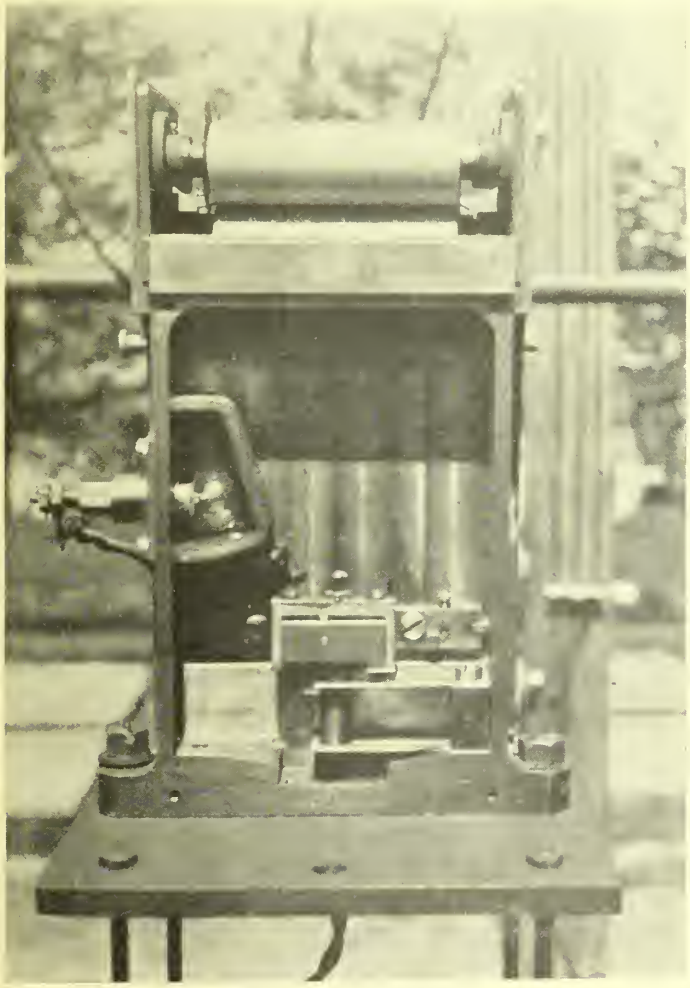

FIG. 1.—Beam accelerometer

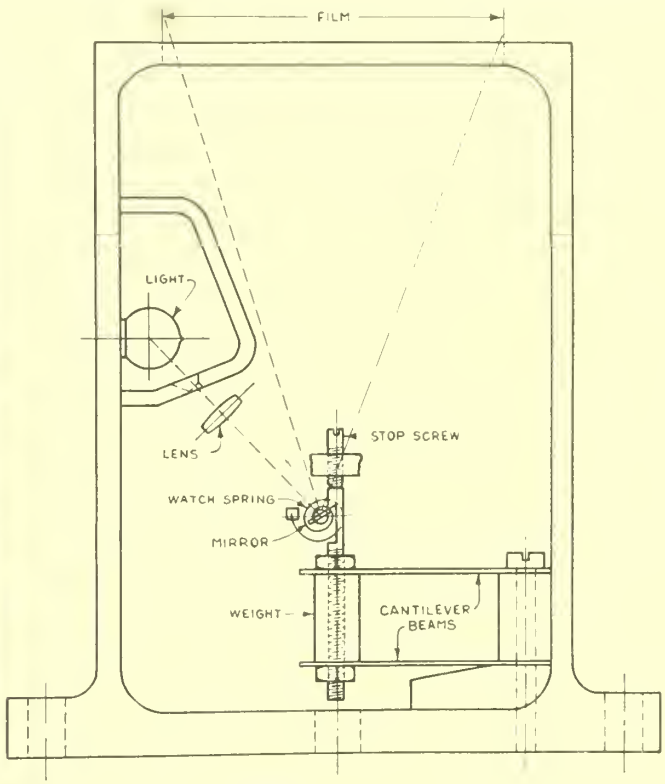

FIG. 2.—Diagram of beam accelerometer

As the mass of the various parts of the motor truck is readily determinable the problem has centered about the measurement of the deceleration of the falling wheel, and, as indicated in the article referred to, this problem has now been satisfactorily solved. Two types of accelerometers have been evolved which are believed to be thoroughly scientific in principle. In one type certain practical defects remain which it is thought can be entirely eliminated; the other is believed to be both scientific in principle and wholly adequate as a practical device for the measurement of

[1] See PUBLIC ROADS, Vol. 5, No. 9, November, 1924. The Status of the Impact Tests of the Bureau of Public Roads, p. 11.

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the kind of decelerations encountered in the study of the impacts delivered by moving vehicles to a road surface.

The latter instrument is now in use in the experiments which are being conducted with moving trucks

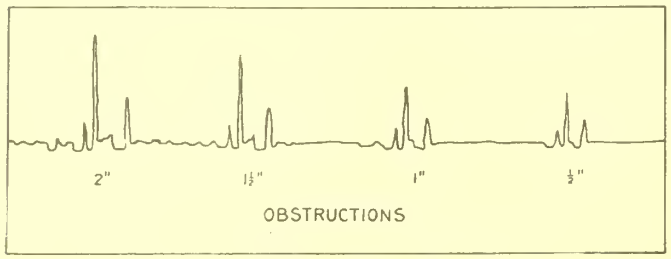


FIG. 3.—A typical beam accelerometer record

operated over actual road surfaces as a result of which it is hoped to determine the magnitude of the impact forces delivered by actual vehicles equipped with various kinds of tires, and the effect of such impacts upon the road surfaces to which they are applied. It is possible that the other type will also be used, and as one or both are certain to play an important part in the subsequent investigations it is desired to record the facts which establish their suitability for the work and the methods used in proving and calibrating them.

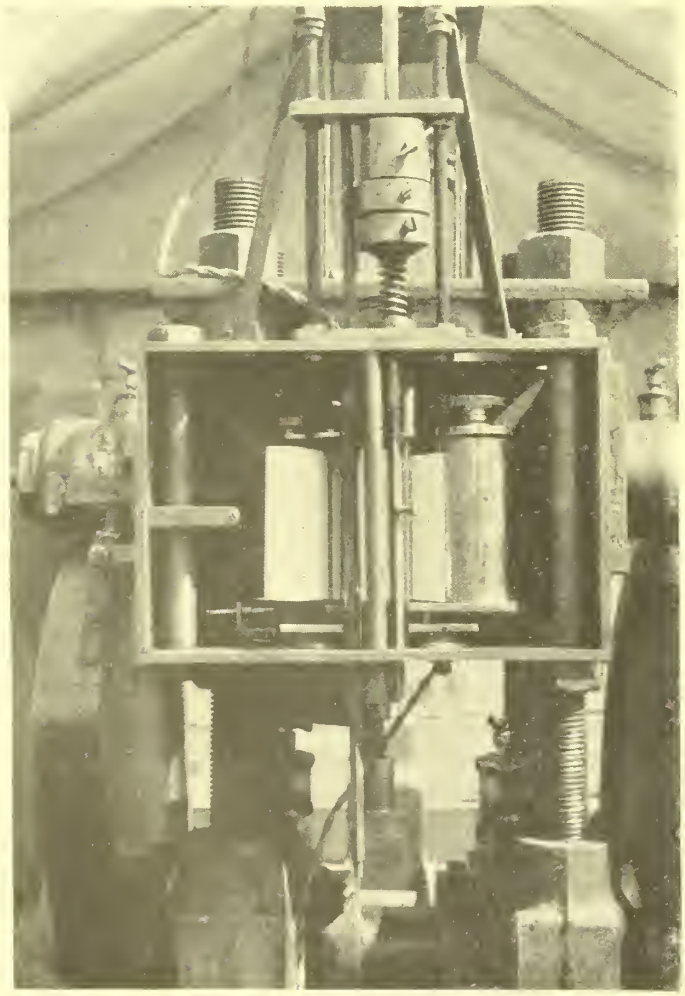


FIG. 4.—Coil spring accelerometer

In addition to the two instuments developed by the bureau and known respectively as the beam type and the coil-spring type, one other accelerometer, known as the sylphon type, was submitted for trial. Although there are a number of other types in existence, varying widely in form and characteristics, all of which have been considered, no other has been found to be suitable for the motor-truck tests.

The beam type of instrument was the first of the two Bureau of Public Roads instruments to be developed. It is shown in Figure 1 and was first described in the Proceedings of the American Society for Test-

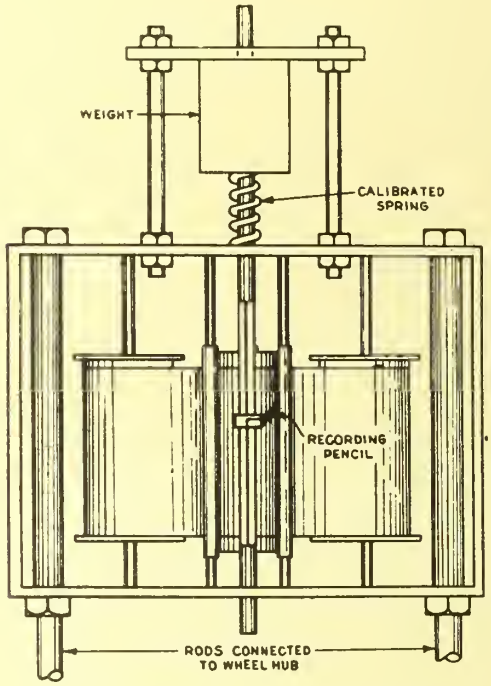


FIG. 5.—Diagram of coil spring accelerometer

ing Materials, 1923, in a paper entitled, "An Accelerometer for Measuring Impact," by E. B. Smith. The second, or coil spring type has been developed more recently and is described in detail in this article for the first time. This instrument makes use of a steel coil spring as the sensitive element instead of the flat steel beam used in the beam type of instrument. The third or "sylphon" type which was submitted to the bureau for trial, employs a sylphon [2] as the sensitive

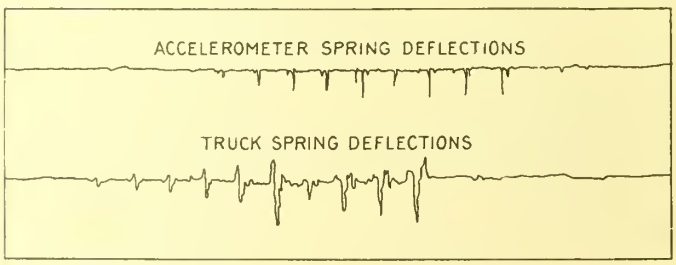


FIG. 6.—Typical record made by coil spring accelerometer

[2] A sylphon is a small accordion-like metal cylinder closed at the ends. As used in the accelerometer the sylphon is filled with a liquid; the impact, which is received on one of the ends, deforms the accordian-like walls, thereby changing the volume of the sylphon and increasing the pressure on the confined liquid. By means of a tubular connection the increased pressure is transmitted to the recording device, which causes movement of the recording point or stylus.

element. It has not yet been developed to the point where it may be regarded as satisfactory for motor truck impact tests.

THE RESULTS OF ACCELEROMETER CALIBRATIONS

These three accelerometers have been subjected to calibration tests in which their indications in response to the same impacts have been compared with the de-

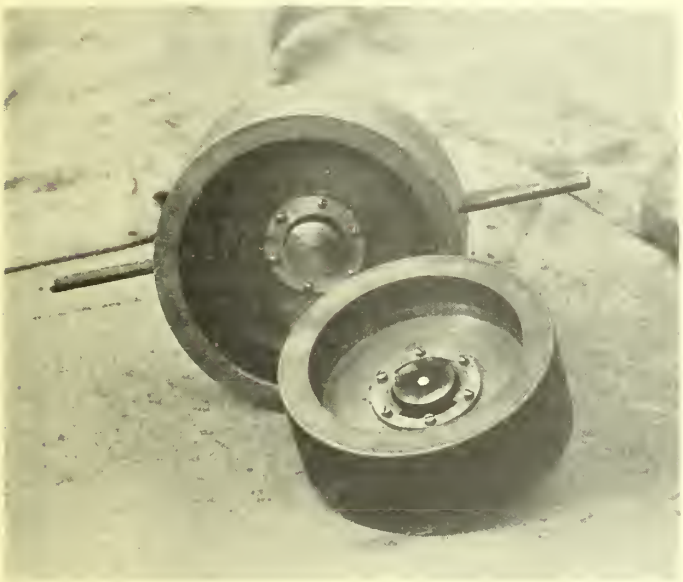


FIG. 7.—Kreuger cell opened to show impression on plane surface

celerations derived from space-time curves of the some impacts, and with each other. The impact computed from the recorded decelerations has also been compared with the force measured directly by means of the Kreuger cell. The aim has been to compare the various types of instruments with respect to their accuracy, consistency, and reliability; to calibrate the instruments which proved satisfactory; to compare impact pressures directly with those computed from the measured decelerations; and finally, to prove that the impact pressure produced by a motor truck in motion can be computed from the measured deceleration, if the masses and spring deflection are known.

The tests show conclusively that the coil spring type of accelerometer is well suited to the measurement of motor truck impact. It consistently indicates the maximum deceleration. The beam type of instrument is thought to be thoroughly scientific in principle, but there are certain defects in its design which make it a less consistent instrument than the coil spring type. The sylphon type has not yet been developed to the point where it may be said to be satisfactory for motor truck impact tests. As an incident of the calibration tests it has also been definitely proved that maximum impact pressures can be computed from space time-curves, made with the apparatus used, with sufficient accuracy for all practical purposes; and that the Kreuger cell indicates the true maximum impact pressure.

THE INSTRUMENTS TESTED

Although the procedure followed in making these tests was comparatively simple, the apparatus used was more or less complicated, and it is thought that familiarity with the latter will help to clarify a description of the former. Therefore, the apparatus will first be described.

Beam accelerometer.—This instrument consists of a flat steel beam, either fixed at the ends or cantilever. A small mass is attached to the beam, and the impressed acceleration, acting on this mass, produces force in direct proportion to the acceleration. This force deflects the beam. The deflection of the beam rotates a small mirror and a ray of light reflected from the mirror onto a strip of photographic film records the magnitude of the acceleration. This instrument is shown in the photograph, Figure 1, and diagramatically in Figure 2. A typical record caused by running over obstructions on the road is shown in Figure 3.

Coil-spring accelerometer.—In this instrument a weight supported on a steel coil spring is impelled downward by a deceleration of vertical velocity, and the consequent deflection of the spring is directly proportional to the magnitude of the deceleration. The recording system, consisting of a paper passing under a stylus which is connected directly to the mass or weight, completes the instrument which is shown in the photograph, Figure 4. A diagram of the instrument is shown in Figure 5 and a record made by running over road obstructions is shown in Figure 6.

Sylphon accelerometer.—A weight on the end of a lever applies pressure to a sylphon when a deceleration is experienced. The pressure is transmitted through a tube to a Bourdon tube recording apparatus.

Kreuger cell.—This simple apparatus, devised by Prof. H. Kreuger, of Stockholm, Sweden, furnishes a means of measuring impact pressures directly. It consists of a spherical steel surface in contact with a plane steel surface which has been coated with a very

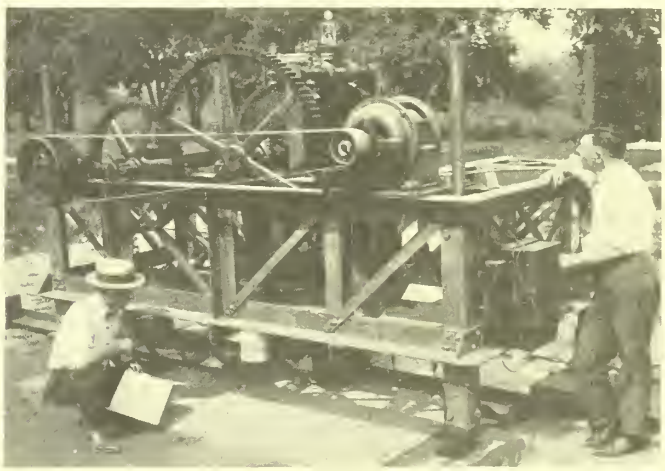


FIG. 8.—The latest type of impact machine

light film of soot from a kerosene lamp. The steel used has a very high elastic limit, and the elastic deformation which occurs under pressure is recorded by the imprint of the maximum area of contact in the smoke on the plane surface. It is only necessary to measure accurately the diameter of the imprint and to refer to the calibration curve to know the maximum pressure which was applied to the cell. Figure 7 shows such a cell opened after a blow, with the imprint visible.

Impact machine.—To reproduce motor-truck impact this impact machine was designed and built by the Bureau of Public Roads. In consists of a truck wheel fastened to the center of a truck spring with a suitable mechanism for raising it and dropping it suddenly. Any condition of wheel, tire, spring, unsprung weight, sprung weight, or height of drop can be had, so that

FIG. 9.—Apparatus for making space-time curves attached to impact machine

practically any impact which any motor truck is capable of producing can be reproduced at will. Figure 8 shows a general view of the impact machine.

Space-time curve apparatus.—Figures 9 and 10 show the apparatus used in these tests to record space-time curves of the movement of the truck wheel of the impact machine during impact. The drum carrying silicated paper is revolved at a uniform rate by an electric motor operating through a worm. The speed of the paper is determined from the record on the paper made by a stylus actuated by an electric tuning fork of known period. The stylus which traces the space-time curve is fastened directly to the hub of the wheel, special precaution being taken to eliminate all lost motion from the system. The whole apparatus is supported by a rigid mounting which is independent of the machine and of the base on which the blow is struck.

THE METHODS USED IN THE CALIBRATIONS

Most of the tests were conducted on the impact machine, sufficient work being done on the road with an actual motor truck to prove that the methods developed permitted the accurate measurement of actual motor-truck impact.

The method used on the impact machine was as follows:

Sprung and unsprung weights corresponding to a standard truck were reproduced. The machine was set up over the center of an 8-inch concrete slab. A steel plate was leveled in grout on the surface of the slab and directly under the truck wheel. On this plate was placed the Kreuger cell. The accelerometers under test were mounted on the plunger of the impact machine, the plunger being the rigid frame which is attached to and moves with the truck wheel. The space-time curve apparatus was then attached to the hub of the wheel. A diagrammatic representation of the arrangement of the apparatus is shown in Figure 10. With all the apparatus in readiness a low drop was made. Each accelerometer registered a certain deflection or reading. These were recorded. The diameter of the imprint on the Kreuger cell was read with a microscope, containing a scale divided in tenths of millimeters, and recorded. The point of contact between the truck tire and the Kreuger cell and the position of the wheel at rest under its normal static load were marked on the space-time curve by setting the truck wheel in each of these positions and then revolving the drum of the space-time curve apparatus. The paper was removed from the drum and marked with the date, test number, etc. This completed the field work for one test. A large number of such tests were made covering a wide range of accelerations.

The tests with the motor truck consisted in running the truck over obstructions so arranged that, when the truck wheel dropped after striking the obstruction, the blow was delivered on the head of a jack containing a Kreuger cell and specially emplaced in the road, as shown in Figure 11. The accelerometers were rigidly attached to the unsprung portion of the truck, one being fastened to the axle at the spring connection, the other to the hub of the wheel. There was

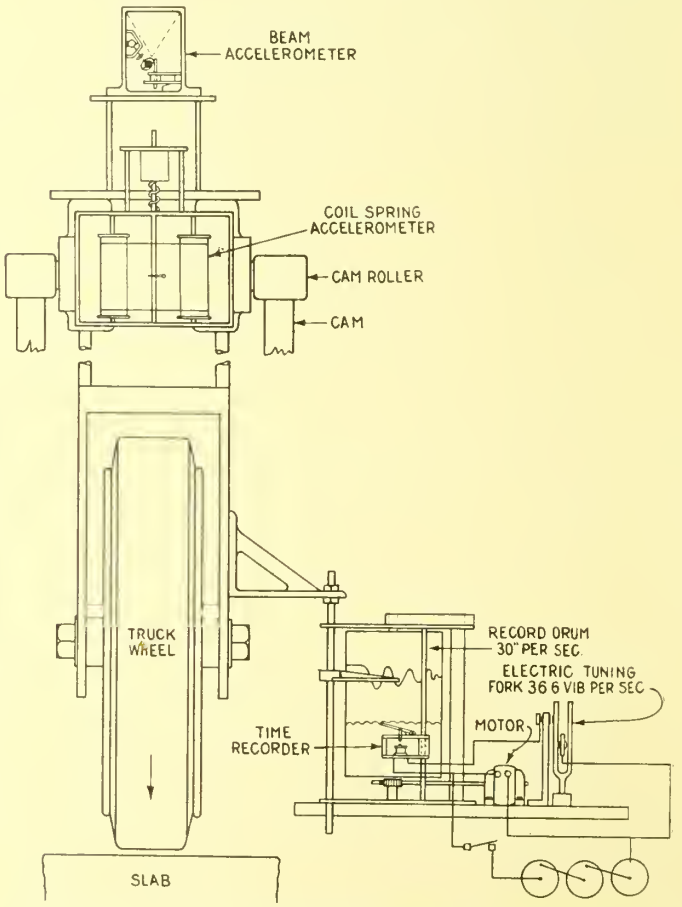


FIG. 10.—Diagram of set-up of accelerometers and space-time apparatus on the impact machine

also a mechanism for recording continuously the deflection of the truck spring. Figure 12 shows the truck equipped with this apparatus, ready for a test run. The data taken consisted of the maximum pressure recorded by the Kreuger cell, the readings of the

two accelerometers and the deflection of the truck spring at the instant of impact.

By running the truck at various speeds and over obstructions of varying heights the desired range of accelerations was obtained.

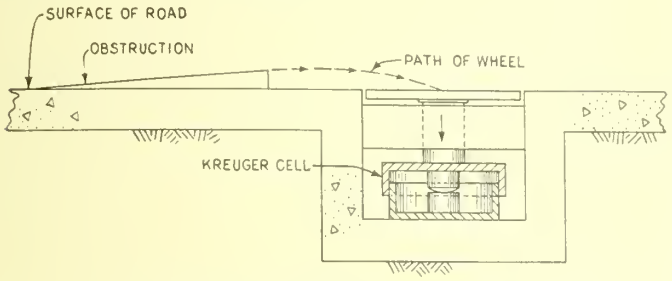


FIG. 11.—Diagram of Kreuger cell emplacement in road

THE ANALYSIS OF THE SPACE-TIME CURVES

If the motion of a body in space, with respect to time is known, its velocity at any point or at any time can be determined. Similarly, if the velocity of the body with respect to time is known, the acceleration at any point or at any time can be determined. From the fundamental laws of motion,

$$dv\ (\text{velocity}) = \frac{ds}{dt}\ \begin{matrix}(\text{space})\\(\text{time})\end{matrix}$$

and

$$\text{acceleration} = \frac{dv}{dt} = \frac{d^2s}{dt^2}$$

The space-time curve is a record of the motion of the mass producing the impact, with respect to time and from it may be obtained a curve showing the changes in velocity during that time, or further, the acceleration values obtaining through those changes.

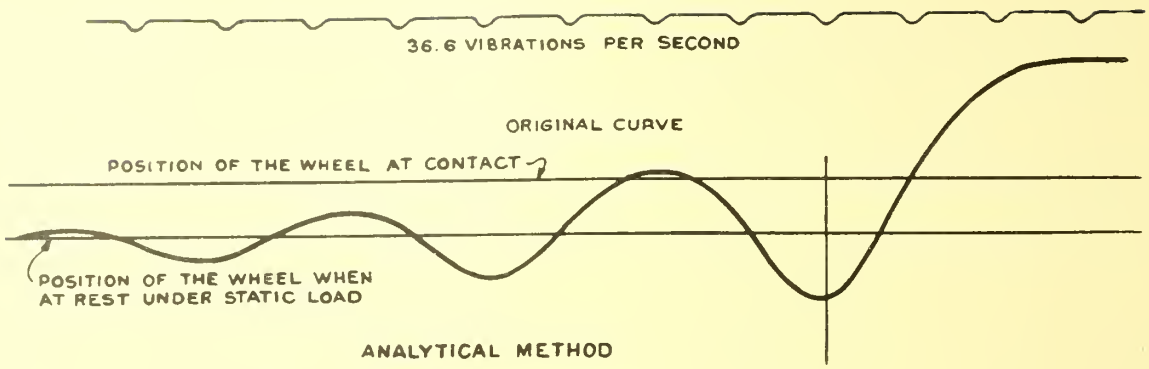
Those who have worked with space-time curves realize the difficulties and errors which will be encountered in their use. It is necessary that the curve represent with absolute fidelity the motion of the body in space, and it is essential that an accurate determination of time be made. The more closely these requirements are met the more accurate will be the result, and if either is seriously in error the result will be quite unusable. For this reason great care was exercised in making the curves used in this calibration test. Two and usually three curves were made for each height of drop used and any tests that appeared doubtful were repeated. Although there is still some variation, the results are, on the whole, very good and prove conclusively that, with due care, the space-time curve can be used to great advantage.

It was found that greater accuracy could be obtained by enlarging the space-time curves and analyzing the enlarged curve. For this work a precision pantograph was used and the curves were enlarged five times. The pantograph was equipped with a prick point and as soon as the guide point was accurately set over a point on the original curve a point was lightly pricked in a sheet of silicated paper for the enlarged curve. The guide point was moved slightly to a new point on the original curve and the corresponding point pricked on the enlarged curve. The enlarged curve finally appeared as a line of very fine holes in the silicated paper. These holes were then connected with a line using a very fine brass point for a pencil and an irregular curve as a guide. All work was done under a magnifying lens and very good results were obtained.

Having the enlarged space-time curve the next step was to determine the maximum acceleration. One method, which was used principally as a check in these tests, is that of graphical differentiation. A series of tangent lines was drawn to the space-time curve at points around its lower extremity as it is at this time that the maximum pressure occurs. These tangents represent velocity at the point of tangency and from them a velocity-time curve was drawn. These velocity curves all showed that, for a certain length of time immediately preceding the final stopping of the truck wheel, the velocity changes at a uniform rate. In other words the acceleration reaches a maximum value and holds it for an appreciable period of time. This had been indicated in all the previous work which had been done by the Bureau of Public Roads with space-time curves of motor truck impact where rubber-tire cushioning was used.

The fact that the lower portion of the space-time curve is parabolic in form led to the use of another method for determining the maximum acceleration. A series of horizontal lines were drawn through the lower part of the curve. A tangent was drawn to the

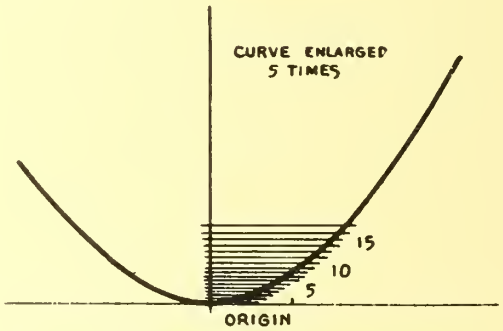
FIG. 12.—Truck equipped with accelerometer and apparatus for recording deflection of truck spring


36.6 VIBRATIONS PER SECOND
ORIGINAL CURVE
POSITION OF THE WHEEL AT CONTACT
POSITION OF THE WHEEL WHEN AT REST UNDER STATIC LOAD

ANALYTICAL METHOD

COORDINATES

PT.	SPACE IN FEET = S	TIME IN SECONDS = t	t^2
1	.000400	.001713	.00000294
2	.000733	.002220	.00000494
3	.001333	.002856	.00000815
4	.001933	.003428	.00001173
5	.002500	.003860	.00001488
6	.003000	.004251	.00001808
7	.003666	.004690	.00002203
8	.004333	.005078	.00002575
	$\Sigma S = .017898$		$\Sigma t^2 = .00010850$
9	.005065	.005520	.00003043
10	.005800	.005900	.00003480
11	.006666	.006320	.00003990
12	.007500	.006725	.00004520
13	.008500	.007180	.00005150
14	.009600	.007615	.00005790
15	.010670	.008020	.00006425
16	.011670	.008400	.00007050
17	.012900	.008820	.00007760
	$\Sigma S = .078371$		$\Sigma t^2 = .00047208$



GENERAL FORM $S = A + Bt^2$

SUBSTITUTING $.017898 = 8A + .00010850\ B$

$.078371 = 9A + .00047208\ B$

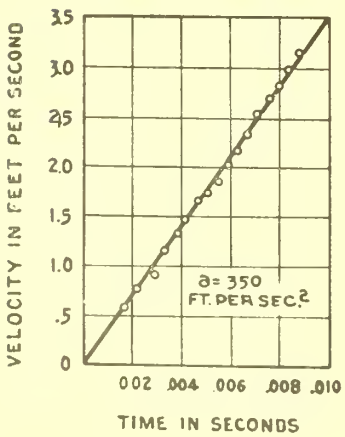
SOLVING $B = 166.3$ $A = .0001$

EQUATION OF CURVE, $S = .0001 + 168.3t^2$

$a = \frac{d^2S}{dt^2} = 332.6$

GRAPHICAL METHOD

TANGENTS



PT.	y	x	VELOCITY FT.-SEC.	TIME IN SEC = t
1	.0108	.01890	.572	.00171
2	.0128	.01680	.762	.00222
3	.0147	.01590	.925	.00286
4	.0170	.01450	1.170	.00343
5	.0185	.01380	1.340	.00386
6	.0197	.01340	1.470	.00425
7	.0209	.01280	1.633	.00469
8	.0220	.01260	1.745	.00508
9	.0230	.01220	1.885	.00552
10	.0240	.01195	2.010	.00590
11	.0250	.01150	2.172	.00632
12	.0258	.01105	2.343	.00673
13	.0271	.01066	2.540	.00718
14	.0279	.01038	2.690	.00762
15	.0285	.01006	2.830	.00802
16	.0293	.00979	2.993	.00840
17	.0299	.00950	3.150	.00882

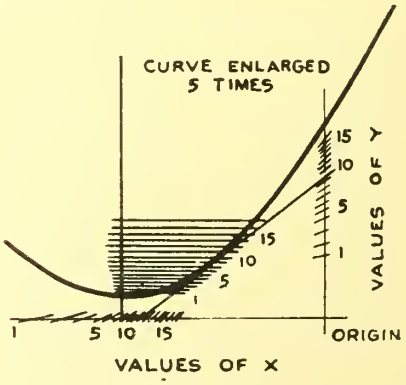


FIG. 13.—Graphical and analytical solutions of space-time curves

tip of the curve and at the point of tangency a perpendicular was erected. Using this point of tangency as an origin the coordinates in terms of space and time were measured for each point where a horizontal line intersected the curve. The number of points taken was usually from 15 to 20. Using these coordinates, expressed in feet and in seconds, the equation of that portion of the curve covered by these points was determined by the method of averages. Having the equation of the curve it is only necessary to differentiate twice to obtain the desired acceleration. A space-time curve and its solution by both methods is shown in Figure 13. Table 1 gives the test data for all the tests on the impact machine for which the space-time curve acceleration values were computed.

TABLE 1.—*Test data for tests in which space-time curves were computed*

Height of fall		Rubber deflection		Acceleration computed from space-time curve	Accelerometer readings			Kreuger cell	
Total	Free	Static	Impact		Beam No. 6	Coil spring	Syphon	Diameter	Equivalent static pressure
Inches	*Inches*	*Inches*	*Inches*	*Feet per sec. per sec.*				*mm.*	*Pounds*
0.665	0.010	0.52	0.655	110	-------	0.10	0.07	7.20	8,300
.755	.035	.51	.72	106	0.23	.11	-------	7.76	10,800
.795	.055	.54	.72	112	-------	.12	-------	-------	-------
1.190	.350	.53	.84	183	.48	.18	-------	8.55	14,700
1.620	.64	.53	.98	252	.67	.25	-------	9.12	17,700
1.65	.66	.55	.99	254	-------	.25	-------	-------	-------
1.65	.66	.56	.99	262	-------	.25	-------	-------	-------
1.70	.71	.52	.99	244	.60	.26	.15	9.05	17,800
1.71	.71	.52	1.00	265	.61	.26	.16	9.14	18,400
1.71	.70	.55	1.00	273	.63	.26	.15	9.16	17,800
1.91	.84	.53	1.07	278	.69	.28	-------	9.25	18,300
1.92	.85	.54	1.07	286	.70	.28	.15	9.32	18,800
1.92	.86	.53	1.06	309	.70	.28	.16	9.53	21,000
2.05	.98	.52	1.07	299	.86	.30	-------	9.65	21,000
2.11	1.01	.54	1.11	341	-------	.33	-------	-------	-------
2.13	1.01	.54	1.12	330	.78	.32	-------	9.56	20,400
2.15	1.01	.54	1.14	336	.79	.33	.18	9.85	22,600
2.16	1.02	.54	1.14	337	.78	.33	.18	9.88	22,700
2.16	1.05	.52	1.10	322	-------	.32	.19	9.76	22,700
2.17	1.06	.52	1.11	357	.79	.33	.20	9.80	23,100
2.27	1.12	.53	1.15	364	.94	.34	-------	9.94	23,700
2.33	1.16	.55	1.17	383	.90	.36	.17	10.05	24,200
2.33	1.16	.55	1.17	377	.88	.36	.16	10.06	24,250
2.34	1.17	.55	1.17	373	.88	.36	.16	10.00	23,900
2.34	1.17	.55	1.17	375	-------	.35	.18	10.02	24,900
2.34	1.17	.55	1.17	375	.86	.36	.18	10.10	25,500
2.34	1.17	.55	1.17	378	.86	.35	.18	10.12	25,700
2.47	1.28	.53	1.19	410	1.00	.39	-------	10.13	24,900
2.50	1.30	.54	1.20	401	.96	.38	-------	10.13	24,900
2.55	1.36	.53	1.19	409	-------	.40	.18	10.10	24,500
2.55	1.34	.54	1.21	422	.93	.40	.19	10.08	24,400
2.55	1.34	.54	1.21	403	.92	.40	.17	10.12	24,800
2.59	1.37	.53	1.22	395	.96	.39	.21	10.12	25,700
2.59	1.38	.53	1.21	424	.96	.39	.17	10.42	28,300
2.69	1.47	.52	1.22	463	-------	.43	-------	10.23	25,700
2.69	1.46	.53	1.23	465	-------	.43	-------	10.18	25,200
2.76	1.50	.54	1.25	469	-------	.45	.18	10.44	27,500
2.75	1.49	.55	1.25	444	1.11	.45	.19	10.35	26,700
2.93	1.66	.52	1.27	473	1.10	.45	.20	10.58	28,600
2.98	1.70	.53	1.28	472	1.22	.48	.21	10.63	29,300
2.99	1.70	.52	1.29	482	1.27	.49	.20	10.73	30,300

METHOD OF COMPUTING MAXIMUM IMPACT PRESSURE FROM MEASURED ACCELERATION

The fundamental formula, force = mass×acceleration, must be somewhat modified to suit the case of a motor-truck impact. The total maximum pressure exerted on the pavement by the truck wheel during impact will consist of three parts and the formula may be written—

$$F = ma + mg + Mg$$

where F = maximum pressure during impact.

m = the unsprung weight of the truck (one rear wheel) divided by 32.2.

M = the sprung weight of the truck acting on one rear wheel divided by 32.2.

a = the acceleration indicated by the accelerometer.

g = the acceleration due to gravity (taken as 32.2 feet per second per second).

When the truck wheel is not in contact with the ground, the spring pressure forces a wider separation of the sprung and unsprung parts, thereby reducing the spring pressure. When, at the instant of impact, the wheel strikes the ground, the spring is again deflected and the deflection increases as the downward motion of the sprung and unsprung parts continues for a time after initial contact of the wheel. It is some time after the total pressure has passed its maximum value before the spring is deflected by an amount equal to its normal deflection under the static weight of the sprung load; i. e., before the full weight of the sprung portion (Mg) comes into effect. The result is that a further modification of the above equation must be made since the third term of the above equation is always less than Mg. However, the deflection of the truck spring at the moment when the impact is a maximum is a measure of the pressure existing at the instant, so that instead of Mg the third term more properly becomes P, the residual pressure exerted by the spring at the instant the unsprung mass has reached its maximum deceleration. The formula then becomes

$$F = m(a+g)+P$$

and this form is used in computing impact from measured accelerations during these tests.

As an example suppose the accelerometer on a certain drop of the impact machine records a deflection of 0.40 inch, m=53.8 and Mg=3,900 pounds.

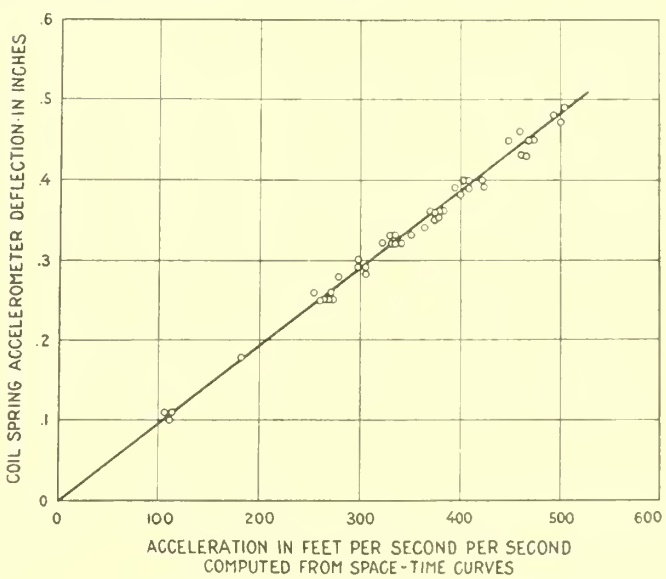


FIG. 14.—Calibration curve of coil spring accelerometer

From the calibration curve (fig. 14) the corresponding acceleration is found to be 416 feet per second per second.

At the moment of maximum impact the spring deflection is less than the static value by 0.74 inches, and

as an inch of spring deflection corresponds to a pressure of 1,800 pounds,

P becomes $3{,}900 - .74(1{,}800) = 2{,}560$ pounds

and $F = 53.8\ (416 + 32.2) + 2{,}560 = 26{,}673$ pounds

The average maximum pressure recorded by the Kreuger cell for an accelerometer reading of 0.40 inch is 25,400 pounds (fig. 15).

THE CHARACTERISTICS OF THE VARIOUS ACCELEROMETERS

Beam accelerometer.—It is thought that this instrument is thoroughly scientific in principle and can be made to serve many useful purposes in the measurement of impact. It is not perfect, and certain changes have suggested themselves during these tests.

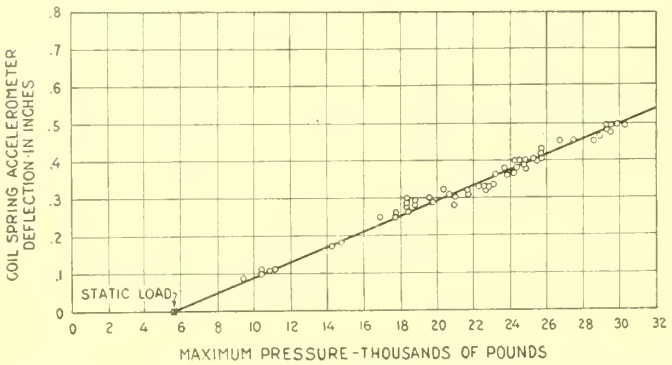


FIG. 15.—Relation of coil spring accelerometer readings and Kreuger cell measurements

One of the first instruments used was fitted with a pair of thin beams fixed at both ends. The deflections or readings for the same conditions of drop seemed to vary from day to day. As the beams were of steel and the mounting was of brass, it was suspected that perhaps temperature was the cause of the variation. A test was run, using static loads to deflect the beam. By varying the temperature of the surrounding air it was found that for a given load considerable variation in deflection resulted. Brass expands roughly 50 per cent more than steel under a given change in temperature so that as the temperature was raised the steel beam became stressed in tension. In this condition the deflection for a given load would be smaller. This was the case during the temperature test. Stiffer beams would not show this effect to such a great extent. A similar temperature test was run on an instrument fitted with cantilever beams and it was found impossible to change the deflection of the light ray for a given load by changing the temperature of the surrounding air as much as 35° F. Thus it was felt that a simple solution of this difficulty had been found. If the other type of beam is preferred, then the beam and mounting should be made of the same material.

It was also found that those instruments which were fitted with a leaf spring to keep the mirror in contact with the hook were inclined to be insensitive to low accelerations because of the resistance of this spring. One of the accelerometers is fitted with a spiral hair spring and this gives a much better curve, i. e., it is a more sensitive instrument than the others.

Considerable trouble was experienced with the electric light bulbs. One trouble was that the bulbs tended to work loose under constant vibration. This was remedied by locking them in the socket with a set screw. The ideal bulb for this instrument would be one with a short straight filament, but so far none has been found.

All of the indications of the test are that the instrument as corrected to eliminate the effect of temperature is consistent and reliable. The accuracy with which the records can be read depends on the sharpness of the record, and this is affected by slight vibration of the lamp filament. An especially designed light bulb would improve this considerably. It is believed that the instrument is very sensitive when properly adjusted and records faithfully the impressed accelerations.

As in the study of motor truck impact, the negative accelerations are the only ones of interest, a stop was provided which keeps the beam from deflection upward beyond its normal position. With such a stop it is not necessary to damp the beam as it can not oscillate in its natural period.

There is a disadvantage in the use of photographic film in that the data are not available until the film is developed, and this work requires another man and special equipment, such as a dark room, etc. This may be of deciding importance in work where speed is the paramount consideration.

The photographic attachment functioned satisfactorily throughout the tests. It was found advisable to use a cloth hood over it, as at times external light got in and injured the record.

Sylphon accelerometer.—This instrument was submitted at the time these calibration tests were about to begin and was tested in its original form without attempt at improvement or development. It is believed that it could be developed into an instrument satisfactory for the measurement of accelerations up to a certain magnitude. In its present state it is quite erratic, and the probable error in the readings is large, too large to give anything like the accuracy desired in these tests. It was found that the vibration of the connecting tube produced comparatively large movements of the recording pen. There was also indication that the deflection was affected by temperature, although no tests were run to prove this.

COIL-SPRING TYPE BEST FOR IMPACT STUDIES

Coil spring accelerometer.—For several reasons this type of instrument has shown itself to be better suited to the measurement of motor truck impact than either of the others. In the first place, the data are immediately available, which is a decided advantage. In the second place, by adding another stylus connected to the truck body, the spring deflection is continuously recorded on the same paper so that the records are simplified. Furthermore, from the very nature of the instrument it can not record accelerations which occur during movements of small magnitude. The movement of the accelerated mass during which a given acceleration is attained must always be greater than the movement or deflection of the sensitive element of the accelerometer in response to this acceleration. For instance, an acceleration caused by the impact of two metal surfaces may reach a magnitude of, say, 100 feet per second per second in a total movement of two or three thousandths of an inch. The record of the coil spring accelerometer can not be greater than this total movement so no record will

be made. In motor-truck impact when rubber tires are used the acceleration will always take place during a considerable movement of the unsprung mass, due to rubber deformation. It has been found that the coil-spring type of instrument will record only those accelerations which are occurring in this unsprung mass and will not show an appreciable record from those other accelerations which may occur from mechanical vibration of metal on metal. Some difficulty was experienced with the beam-type accelerometer from this cause. Due to the very small movement of the beam it is sensitive to accelerations which occur during very small movements, such as those due to mechanical vibration, and it was found, both in the tests on the impact machine and on the trucks where a connecting link was used between the instrument and the truck wheel, that such vibration caused a variety of accelerations to be recorded. On the trucks this difficulty was overcome by mounting the beam-type accelerometer directly on the axle over the spring connection. On the impact machine a special stand was built which interposed four slender columns between the base of the instrument and the crosshead of the machine on which it was mounted. These columns served to cushion the instrument against lateral vibrations and largely eliminated the effect of vibration. The magnitude of the movement during which an acceleration occurs is a very important consideration which should not be lost sight of wherever accelerometers of any type are used.

Kreuger cell.—The chief difficulty experienced in the use of the Kreuger cell is in the accurate measurement of the diameter of the imprint left on the plane surface after the blow. Due to the thickness of the film of carbon, there are two rings marking the circumference of the record, the inner one being the actual area of contact and being composed of compressed carbon, the outer one appearing as a "halo" around the area of contact due to the carbon being pushed away from the area of contact. If the film of carbon is very thin the "halo" band will be very narrow and vice versa. The method of measurement used in these tests was as follows: A microscope containing a scale graduated in tenths of millimeters was placed over the record and the diameter of the area of contact estimated to the nearest 0.05 millimeter. Two diameters were taken at right angles to each other and averaged. This was done by two observers. If the readings all checked within about 0.1 millimeter the average of the four readings was taken as the diameter. Otherwise all readings were checked. If the edge of the circle of contact always appeared as a sharp line, there would be no trouble in making this determination, but very often the edge of this area is quite indefinite. It was found in calibrating that errors of between 5 and 10 per cent would sometimes be made in pressure values, although both observers had checked fairly well on the diameter of the area of contact. Sometimes, however, the records are very clear and close determinations can be made. It has been found that a very light smoke film and warm, dry steel surfaces improve the quality of the record. This instrument furnishes a most valuable addition to the instruments for the measurement of impact.

THE ROAD TESTS

The ultimate aim of this calibration was to determine whether or not it is possible to compute impact pressures from measured accelerations on a motor truck in motion if the sprung and unsprung weights are known and the spring deflection is measured. It has been conclusively demonstrated that such impact pressures can be computed with a satisfactory degree of accuracy.

Table 2 shows the data of 39 road tests where the computed pressure is compared with the pressure indicated by the Kreuger cell. These comparisons were made for loaded and unloaded trucks, four heights of drop and four speeds. With speeds greater than 10 miles per hour it is very difficult to cause the truck to land squarely on the Kreuger cell. But it is thought that the data show that the accuracy of the method is independent of the speed of the truck.

The accelerations range from 0 to 400 feet per second and the spring pressure P varies from −200 pounds to +2,000 pounds. In all cases there is a possible variation of perhaps 10 per cent in the pressures measured by the Kreuger cell. Considering these things it is thought that the two measurements check very well.

TABLE 2.—*Comparison of computed and measured impact pressure during motor truck impact*

Example:

$$m=37.3$$

Accelerometer reading 0.236=245 feet per second per second.
Spring deflection 0.658" corresponding value of P=755 pounds.
$P=m(a+g)+P=37.3(245+32.3)+755=11{,}094$ pounds.
P measured by Krugergesell=10,500 pounds.

Test No.	Sprung weight	Unsprung weight	Spring deflection	P	Acceleration	Pressure by computation	Pressure by Kreuger cell	Speed of truck
	Pounds	*Pounds*	*Inches*	*Pounds*	*Feet per sec. per sec.*	*Pounds*	*Pounds*	*Miles per hour*
1	2,075	1,200	0.03	2,015	0	3,215	3,500	2
2	2,075	1,200	.02	2,035	0	3,235	3,600	2
3	2,075	1,200	.09	1,900	41	4,630	3,800	4
4	2,075	1,200	.20	1,675	98	6,535	6,550	4
5	2,075	1,200	.37	1,330	118	6,930	6,500	4
6	2,075	1,200	.41	1,270	162	8,510	8,400	4
7	2,075	1,200	.70	675	256	11,425	11,450	4
8	2,075	1,200	.66	755	262	11,715	11,900	4
9	2,075	1,200	.70	675	254	11,350	10,500	4
10	2,075	1,200	.66	755	245	11,105	10,500	4
11	2,075	1,200	1.22	−40	259	10,810	11,400	4
12	2,075	1,200	1.04	320	255	11,050	11,700	4
13	2,075	1,200	.49	1,090	143	7,625	6,950	7
14	2,075	1,200	.44	1,200	121	6,910	6,900	7
15	2,075	1,200	1.03	350	243	10,625	11,300	7
16	2,075	1,200	1.05	320	250	10,850	11,700	7
17	2,075	1,200	1.28	−160	323	13,090	13,800	7
18	2,075	1,200	.25	1,600	156	8,620	6,700	10
19	2,075	1,200	.27	1,500	127	7,430	6,200	10
20	2,075	1,200	.56	965	197	9,515	9,100	10
21	2,075	1,200	.54	1,000	170	8,550	8,900	10
22	2,075	1,200	.68	1,040	175	8,765	9,500	10
23	2,075	1,200	1.10	200	225	9,800	9,400	10
24	2,075	1,200	1.11	180	225	9,780	11,200	10
25	2,075	1,200	1.22	−40	230	9,740	10,300	10
26	2,075	1,200	1.25	−100	258	10,720	10,700	10
27	2,075	1,200	1.33	−200	265	10,900	10,400	10
28	2,075	1,200	1.28	−150	286	11,730	11,400	10
29	2,075	1,200	1.32	−200	255	10,510	10,700	10
30	3,800	1,200	1.74	320	385	15,870	15,900	10
31	3,800	1,200	1.89	0	345	14,050	15,600	10
32	3,800	1,200	2.10	−400	310	12,380	14,600	10
33	3,800	1,200	2.01	−220	322	12,980	13,700	10
34	3,800	1,200	1.59	620	384	16,170	15,900	10
35	3,800	1,200	1.80	200	365	14,975	15,600	10
36	3,800	1,200	1.44	920	416	17,620	15,400	10
37	3,800	1,200	1.62	560	364	15,300	14,800	10
38	3,800	1,200	1.50	800	385	16,300	15,200	10

HIGHWAY TRAFFIC IN CALIFORNIA

SOME APPLICATIONS OF THE TRAFFIC COUNTS MADE BY THE BUREAU OF PUBLIC ROADS

By L. I. HEWES, Deputy Chief Engineer, United States Bureau of Public Roads

THE position of California with respect to highway transportation is unique. The very large motor-vehicle registration, the outdoor climate, and the well-developed highway system of the State combine to make traffic exceptional. The measure of this traffic has been taken and will continue to be taken, and these things combine to make the experience of this State of general interest and value to highway engineers and highway economists. An indefinite number of statistical relations from the state-wide traffic counts might be set up, but it is the object of this paper to present only the major features of the traffic development and their economic significance.

Before 1920 there had never been a State highway traffic count in California. That year, in connection with a study of the State highway system made by the Bureau of Public Roads, a preliminary traffic count was made on the basis of one equivalent 16-hour day at 103 stations on 2,150 miles of the main State highways. The count was intended to sample the summer traffic, June 1–November 1, only. It was made between August 7 and October 14, and showed that the average total traffic between 6 a. m. and 10 p. m. was 1,382 vehicles. Of this traffic 12.58 per cent was motor trucks and 2.7 per cent horse-drawn vehicles.

Beginning about 1915, traffic counts had been made by Kern and Los Angeles Counties on selected highways, and they showed a progressive increase in average truck traffic in Kern County from 25 in 1915 to 140 in 1920, and in Los Angeles County from 95 trucks in 1915 to 330 in 1920.

The State census of 1920 was intended principally to determine the usefulness of the State highways and to measure the relation between traffic and the existing condition of 2,150 miles of constructed roads. For each of the roads on which the traffic was observed traffic diagrams or traffic profiles were prepared, these consisting of a plot with State highway routes shown horizontally and daily motor trucks and vehicles vertically, both to suitable scales. The area below the traffic curve on a given route thus indicated the total daily traffic in vehicle-miles. From these daily traffic diagrams for the several routes the total daily use of the State highways was developed. Thus the daily summer traffic in 1920 was estimated as approximately two and one-half million vehicle-miles for the main surfaced highways. From the daily traffic profiles of all routes measured a total estimated movement of traffic of 375,000,000 vehicle-miles was computed for the summer interval alone (June 1 to November 1).

In making this count it was necessary to observe the 8-hour law and instead of one continuous 16-hour day from 6 a. m. to 10 p. m. it was frequently found convenient to build up an equivalent day by two or more shifts. Thus the equivalent-day count represents the assembly and average of 8-hour fractions of from 2 to 12 calendar days at the various stations. Sufficient 24-hour counts were made to check the relation with the 16-hour traffic, and it was found to be 88 per cent of the total for the 24 hours.

A questionnaire was sent to 21,000 truck owners, to which there were 1,930 available replies, which attempted to answer the questions of average distance operated daily, average weight, advantageous speed, and miles per gallon of gas. The questionnaire involved considerable labor and expense, and while it was not satisfactory it had certain value as a check. It showed that the trucks of larger capacity (from 3½ to 7 tons) operated a greater average daily distance and at a reduced speed; that agricultural products constituted the main commodity hauled, with back hauls of groceries, provisions, and general merchandise, but preeminently of building materials and machinery. The questionnaire indicated that the average net load of 1920 was 2.1 tons, and that the inbound tonnage had a ratio to the outbound of 1.38.

In 1920 attention was also given to the passenger bus lines, which had already reached an advanced stage of development. Bus lines are regulated by the State railroad commission under the act of May 1, 1917, as amended May 13, 1919.[1] The commission regulates rates and conditions and requires reports on operations. Under their jurisdiction and protection against unfair and unnecessary competition, and enforcement of schedules, etc., passenger bus business has developed. In 1920 there were 103 independent bus lines, excluding those with insufficient data. The operation of these bus lines involved 3,075 miles of State highways, with an average daily movement of 65,220 vehicle-miles, of which 46,046 were on State highways only. The longest trip then possible by purchase of through transportation tickets over connecting lines was from El Centro, via San Diego, Los Angeles, and San Francisco to Portland, Oreg., a distance of 1,448 miles. Passenger-carrying bus capacities of the larger companies ranged from 8 to 22.

TRAFFIC COUNT OF 1922

At the request of the California highway commission the Bureau of Public Roads cooperated in a second State-wide traffic census in 1922.[2] For purposes of comparison it was essential that traffic be counted at the same 103 stations occupied in 1920. In addition to these, 138 other stations were established which completed the original counts and extended the scope of the count from 2,150 miles to 3,200 miles, thus including graveled portions and some graded mileage in the State system. The traffic was classified as passenger cars and trucks, and this classification was again subdivided into light and heavy. Trucks were recorded as loaded and empty separately. There was also a provision for horse-drawn vehicles, busses, and trailers.

The summer traffic count for 1922 was commenced on August 27, and was completed September 20. This was followed by a winter count, which was begun January 25, 1923, and ended March 3. The average summer traffic in 1922 was found to be 2,037 total vehicles per day, an increase of 47.4 per cent over the corresponding figure of 1920. The winter day average for 1922–23 was 1,578, and the yearly average 1,800.

[1] This law exempted bus lines operating prior to May 1, 1917.
[2] This census was actually completed in 1923.

In addition to the State highway counts, others were made by 24 cooperating counties at 190 stations which, in general, included records of from 6 to 24 days per station, usually for 16 hours. The total number of stations at which traffic was counted in 1922–23 was, therefore, 431. The counts of 135 of the county stations synchronized sufficiently with the State counts to permit the inclusion of the resulting data in the State-wide satistics. The daily summer average of all vehicles on county roads was 722, except in Los Angeles County where it ran up to 5,338. Since the trucks were heavy. About 55 per cent of the total trucks were found to be loaded, in counting, and about 68 per cent in the weighing, but in the weighing empties were neglected when traffic was very dense. Nearly 60 per cent of the loaded trucks were overloaded with reference to the rated capacity of truck. But, of the trucks weighed, 11.7 per cent were overloaded with respect to the 1921 law (700 pounds per lineal inch width of tires), and of these overloads with respect to the law 52.8 per cent could have been avoided by redistribution of the load.

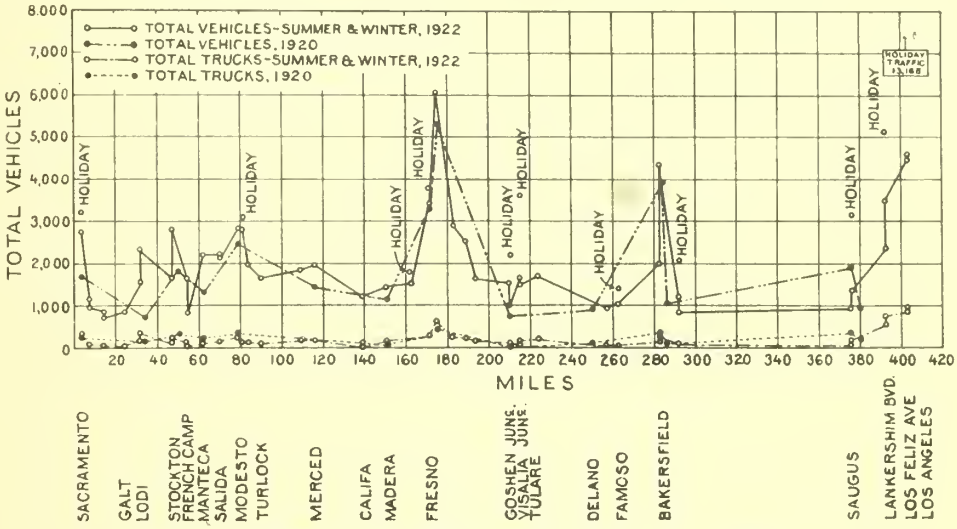


FIG. 1.—Traffic profile of highway from Sacramento to Los Angeles

county stations represent about 2,100 miles of county highways adjoining the State highways, such county highways may be said to carry about 26 per cent of the total traffic recorded.

Samples of the traffic profiles prepared from the 1922 census are shown in Figures 1 and 2. The total annual movement of motor vehicles on all State roads observed (about 3,200 miles), computed from these profiles was 1,370,000,000 vehicle-miles. The daily average traffic on the same 2,150 miles observed in 1920 was approximately 2,750,000 vehicle-miles in 1922, an increase of about 13 per cent over 1920.[3] The 1922 count also represents an equivalent 16-hour day from 6 a. m. to 10 p. m. This 16-hour day, as in 1920, was built up by combinations and averages of about 3,000 eight-hour fractions. For the most part, however, the 16-hour days in 1922–23 were continuous and both the summer and winter average daily counts are representative of an average of approximately four days each.

During the 1922–23 traffic count, trucks were weighed at 80 stations where truck traffic was sufficiently heavy to justify the expense. Approximately 13,000 trucks and 1,100 trailers were weighed. It was found by counting that winter trucking was to summer trucking about as 55 to 100 numerically, and that truck traffic was approximately 10 per cent of all traffic. Trucks classed by counting as heavy (over 2½ tons capacity) were about 38.3 per cent of the total. The weighing statistics indicated, however, that 45.1 per cent of

The average net load in 1922 was 3.3 tons.[4] Trucks of capacity of ½ to 1½ tons constituted 24.5 per cent, and trucks of capacity 2 to 2½ tons, 31.8 per cent; trucks of capacity 3 to 4 tons, 27.2 per cent; and trucks of capacity 5 to 7½ tons, 16.5 per cent, of all the trucks weighed. The 16.5 per cent of large trucks, however, carried 25 per cent of the net loads weighed. It was found that approximately half the truck traffic from a given point fell within a range of approximately 21 miles, but the average haul was 31 miles.

PASSENGER BUS LINES INCREASING

In 1922 there were 710 franchised motor freight and passenger bus lines of all sorts under the control of the railroad commission. Some of the passenger bus lines carried express also and some carried freight. There were 61 class A automotive transportation corporations, having an annual operating revenue exceeding $20,000, of which 17 had revenues exceeding $100,000.[5] Many of the smaller lines have United States mail contracts and carry passengers. The figures for 1922 show that the ratio of operating expenses of the class A lines to operating revenues was 93.3 per cent. Total net operating revenue was in excess of half a million dollars. The average length of bus lines is apparently 41 miles. There appears to be a tendency toward consolidation, the most successful lines acquiring the smaller lines. Bus traffic on some of the highways has become a considerable fraction of the total traffic. Between San Francisco and San Mateo there

[3] The apparent discrepancy between this percentage of increase and the 47.4 per cent of increase in average traffic is due to the much greater accuracy of the 1922 traffic profiles.

[4] Compare with net load of 2.1 tons reported in 1920 by questionnaire.

[5] Most of the lines having an operating income in excess of $100,000 were in operation prior to May 1, 1917.

are 200 busses daily, and about 115 between San Mateo and San Jose. Between Los Angeles and Whittier there are over 200 busses daily and about 130 per day north of Santa Ana. The average rate per passenger-mile is apparently about 5.4 cents. Of 246 bus lines classified as to length of operation, 80, representing the largest class, operated from 25 to 50 miles, and the total length of these lines is approximately 10,500 miles.

Freight trucking lines, in filing applications for certificates of convenience and necessity with the railroad commission, must show proposed rates, schedules, and freight classifications. They usually make their own classifications, but a gradual standardization is being evolved by the larger operators. The present rates vary from 20 cents per ton-mile for short hauls down to 8.3 cents per ton-mile for hauls of about 100 miles.

COMPARISON OF THE 1920 AND 1922 TRAFFIC COUNTS

In connection with the census of 1920 the Bureau of Public Roads attempted to ascertain characteristic or invariant elements connected with California highway transportation. Among these were the seasonal changes in traffic, the weekly periodic variation for which was found a first approximation, the daily variation from hour to hour, the balance of going and coming traffic, and the percentage of traffic between 6 a. m. and 10 p. m. Studies of speed of various classes of vehicles were also made with some useful results. All the 1920 results and deductions have since been refined and checked by the census of 1922.[6]

The seasonal variation which was shown to exist in 1920, but only approximated, was satisfactorily established in 1922 to indicate a ratio of total winter to total summer traffic, approximately as 77 is to 100, with an exception for trucks. Truck travel fell off much more during the winter and the study showed the importance of the harvest season in California. The characteristic weekly variation of traffic in California shows that only 58 per cent of the weekly traffic occurs in the first five days. The balance occurs on Saturday and Sunday. The daily variation of traffic is found to be characterized by a forenoon peak about 10 o'clock, and a greater afternoon peak at 5 o'clock. There is some deviation from the characteristic curve during the winter, when the afternoon peak seems to increase and the forenoon peak to decrease. The daily total traffic variation differs somewhat from that of the truck traffic alone, the two peaks of which tend to be more nearly equalized, and a greater percentage of truck traffic occurs prior to 6 a. m. The going and coming traffic so nearly equalizes (as it should) that in 1922 no separate records of direction were made, and this simplified the counting. It was found in 1922 that about 92 per cent of total traffic occurs between 6 a. m. and 10 p. m., against 88 found in 1920. The higher percentage checks with the counts in several Tennessee counties in 1922. As in 1920, stop-watch speed tests on measured sections of the highway were made in 1923, and compared with the 1920 results without revealing marked differences. The 1922 tests were more accurate and showed a higher average speed. Heavy passenger cars operate with a greater speed, and heavy trucks at the lowest speed, but commercial passenger busses operate at the highest speed of all, approximating 33 miles per hour on an average. Table 1 shows the average week-day traffic at the comparable 103 stations for the two years, 1920 and 1922.

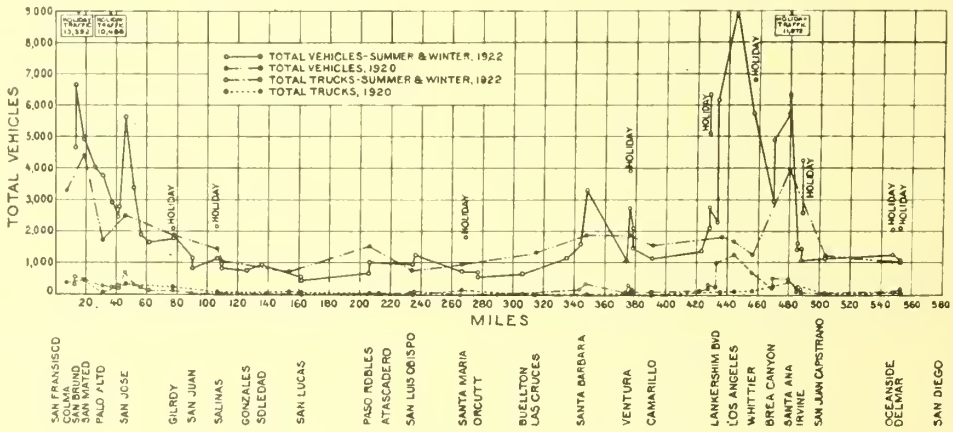


FIG. 2.—Traffic profile of highway from San Francisco to San Diego

TABLE 1.—*Average week-day traffic at 103 stations, 1920 and 1922*

	Number of vehicles					Percentage of total vehicles				
	Summer		Winter 1922	Average day 1922	Counties	Summer		Winter 1922	Average day 1922	Counties
	1920	1922				1920	1922			
						Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Total vehicles	1,382	2,037	1,578	1,800	722	100.0	100.0	100.0	100.0	100.0
Total trucks only	173	231	127	180	83	12.5	11.3	8.0	10.0	11.5
Passenger cars	1,146	1,724	1,388	1,547	604	83.0	84.7	88.0	85.9	83.7
Light trucks, loaded	173	73	38	55	27	12.5	3.6	2.4	3.0	3.7
Light trucks, empty		69	41	56	28		3.4	2.6	3.1	3.9
Heavy trucks, loaded		58	30	44	18		2.8	1.9	2.5	2.5
Heavy trucks, empty		31	18	25	10		1.5	1.1	1.4	1.4
Horse-drawn vehicles	32	24	17	21	17	2.3	1.2	1.1	1.2	2.3
Busses	31	35	31	33	6	2.2	1.7	2.0	1.8	.8
Truck trailers	-----	23	15	19	12	------	1.1	0.9	1.1	1.7

On the 2,150 identical miles covered both in 1920 and 1922 traffic was found to be 98.8 per cent motor driven,

[6] For details of the 1920 count see "Study of California Highway System" by U. S. Bureau of Public Roads, and for 1922 count see "A report of Traffic on State Highways and County Roads in California," 1922, both published by the California Highway Commission.

as against 97.7 per cent in 1920. The increase in total daily traffic was 47.4 per cent, or from a summer average of 1,382 vehicles in 1920 to 2,037 in 1922. In the same interval the automobile registration had increased 48 per cent. On the county roads also 97.7 per cent of all traffic in 1922 was found to be motor driven.

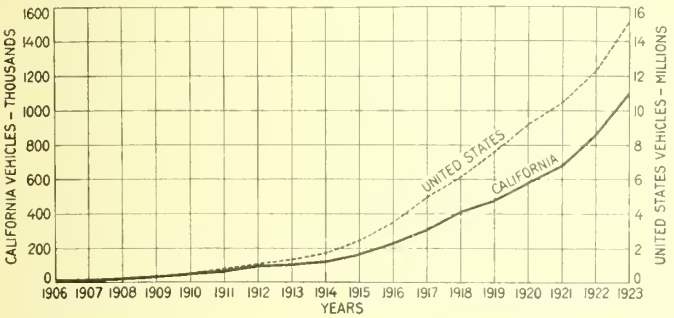


FIG. 3.—Increase in motor vehicle registration in California compared with increase in United States from 1906 to 1923

In 1922 special attention was given to motor-truck data. The results of weighing 13,000 trucks and inquiry at 80 stations show that the ratio of winter to summer trucks is as 55 to 100.[7]

THE CALIFORNIA AND CONNECTICUT TRAFFIC SURVEYS COMPARED

By comparison with the results of the Connecticut traffic census, taken in 1921 and 1922 by the Bureau of Public Roads, it is found that the California truck traffic in general shows characteristics similar to the traffic in Connecticut. In both States manufactured products predominate as the commodity hauled. It is found in both States that the percentage of empty trucks decreases with increases in the length of haul. It is found in California that light trucks (1½ to 2½ tons) were 56.3 per cent of all trucks and carried 40.1 per cent of the total net tonnage, while in Connecticut similar light trucks, which were 66.6 per cent of all trucks, carried 45.3 per cent of all commodities. There are, however, some interesting differences. In California there is a greater tendency for the loads to overrun the rated capacity of the truck. In California the 1922 census indicated that about half the trucks haul less than 21 miles and that the average haul is approximately 31 miles, whereas in Connecticut the average truck haul was only 15.7 miles for Connecticut trucks alone. No particular relation between width of truck body and overloading of the truck was observed in California, although a fairly definite relation was found to exist in Connecticut.

Truck registration data in California in the past have not been satisfactory because commercial vehicles or trucks with pneumatic tires have been registered up to 1924 as automobiles, and only vehicles with solid tires have been classed as trucks. This classification probably accounts for a decrease from 41,689 trucks registered in 1920 to 35,092 in 1921. Registrations for 1922 and 1923, however, show increases to 39,413 and 43,527, respectively. In the 1920 traffic census all trucks were counted as such irrespective of their tire equipment, and the daily average on the highways was found to be 173 at the 103 stations comparable with the 1922 count. In 1922 the truck traffic was found to be 231, exclusive of the pneumatic-tired vehicles, which, in harmony with the registration procedure, were classed as heavy automobiles. It is thought that the small increase shown in truck registration from year to year is probably due to the adoption of pneumatic tires to an increasing extent for all lighter trucks, particularly commercial vehicles of the delivery type, and also to a relatively large increase in this class of commercial vehicles operating in cities. The motor vehicle law as amended in 1923 now provides for registration of commercial vehicles irrespective of their tire equipment as trucks, and up to April, 1924, there had already been registered approximately 106,000 pneumatic-tired trucks and 35,000 solid-tired trucks. This total of 141,000[8] trucks immediately reflects the change in classification over the preceding year.

The progress of California motor-vehicle registration since 1907 is shown in Table 2. The increases in registration shown in the table are compared with the increases in the United States as a whole in Figure 3, and with those of the five leading States in Figure 4. Based on the 1920 census, with conservative allowance for population increase, the number of persons per motor vehicle in California in 1923 was approximately

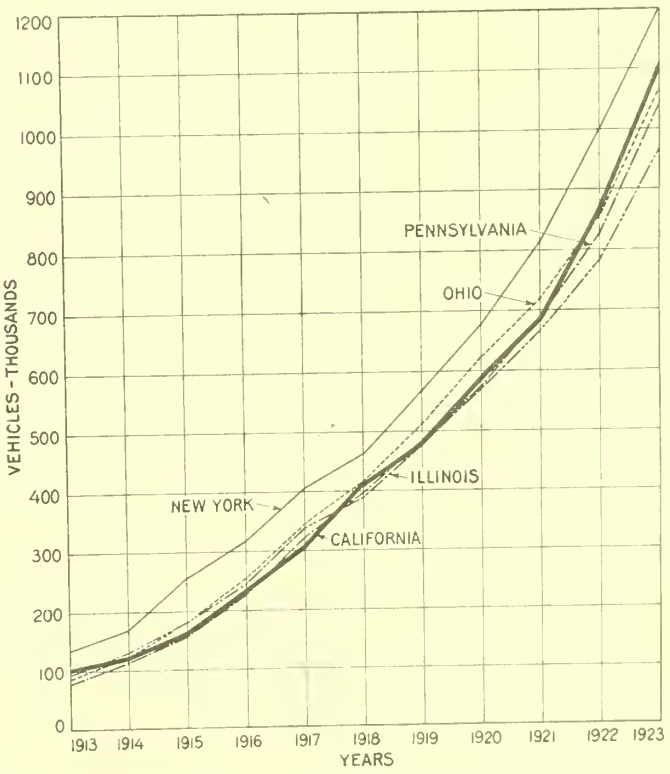


FIG. 4.—Motor vehicle registration in California and five leading States compared

3.5, and the average number of persons per motor vehicle for the entire United States was approximately 7.3. The ratio of motor vehicles to people in California is not exceeded in any other State.

[7] See "A Report of Traffic on State Highways and County Roads in California," published by the California Highway Commission.

[8] Up to July 1, 1924, the registration of trucks was 166,459.

TABLE 2.—*Motor vehicle registration and revenues in California, by years*

Year	Total vehicles			Trucks only			Registration receipts		
	Number [1]	Increase		Number [3]	Increase		Amount	Increase	
		Number	Per cent		Number	Per cent		Amount [4]	Per cent
1907 [2]	10,020								
1907	14,051	4,031	40						
1908	19,561	5,510	39						
1909	28,633	9,072	46						
1910	44,122	15,489	54						
1911	60,779	16,657	38						
1912	91,194	30,415	50						
1913	118,716	27,522	30						
1914	123,504	4,788	4	6,156			$1,333,785		
1915	163,797	40,293	33	8,189	2,033	33	2,027,432	$693,647	52
1916	232,440	68,643	42				2,192,699	165,267	8
1917	306,916	73,476	32				2,846,030	653,331	30
1918	407,761	100,845	33	13,953			3,524,036	678,006	24
1919	477,450	69,689	17				4,468,721	944,685	27
1920	583,623	106,173	22	41,689			5,714,717	1,245,996	28
1921	680,614	96,991	17	35,092	−6,597		6,834,089	1,119,372	19
1922	861,807	181,193	26	39,413	4,321	12	8,384,606	1,550,517	23
1923	1,100,283	238,476	28	43,527	4,114	10	10,608,544	2,223,938	27

[1] Years 1907 to 1913, inclusive, from Report of a Study of the California Highway System, by the U. S. Bureau of Public Roads, p. 121. Years 1914 to 1923, inclusive, from PUBLIC ROADS, Vol. 5, No. 2, April, 1924.

[2] Prior to Apr. 8, 1907.

[3] Years 1914, 1915, and 1918, from Report of a Study of the California Highway System, by the U. S. Bureau of Public Roads, p. 121. Years 1920 to 1923, inclusive, from Division of Motor Vehicles, California.

[4] From PUBLIC ROADS, Vol. 5, No. 2, April, 1924.

STATISTICS INDICATE LIFE OF VEHICLES

Since 1921 the actual number of new motor vehicles sold in the State has been made a matter of record. These statistics accumulated for a sufficient period and coupled with the registration statistics will furnish the most reliable index of the average life of the motor vehicles. Figure 5 illustrates the method. The total supply curve for the years 1921 to 1923, inclusive, represents the actual number of new vehicles sold in the State. For the years prior to 1921 the points have been obtained by computation in the following manner: From the total annual production of the United States there have been allocated to California a number of new vehicles, fixed by the ratio of California's registration to the total registration of the country for the corresponding year. The total discharge curve is obtained by subtracting from the total supply the California registrations for each corresponding year. Both curves are cumulative. They represent the total number of vehicles supplied and discharged from service at a given date. By construction, the vertical distance between them represents the registration for each year; and the horizontal distance intercepted represents, for any given year, the average life of the vehicles produced during the year. The approximate curves of Figure 5 indicate that the vehicles sold in 1915 have had an average life of about 8.5 years. There is no certainty that this figure represents their actual life, but a continuation of the actual record of sales will soon yield figures of real dependability. Such figures, indicative of the life and rate of depreciation of vehicles, are essential to the determination of vehicle operating costs which in turn are required as a means of estimating the economic worth of highway improvement and the share of the cost of such improvement which properly may be laid upon the motor vehicle.

VEHICLE OPERATING COSTS AND TAX RATES IN RELATION TO THE COSTS OF HIGHWAY IMPROVEMENT

There is a growing conviction that highway traffic must be sufficiently taxed to pay the cost of maintenance and upkeep and at least a part of the cost of highway construction.

Taxation for such a purpose is essentially rate making for highway transportation and should follow closely the principles of rate making applying to other public utilities. A method by which the tax return can be adjusted to highway costs and the validity of the latter determined is illustrated by the following analysis.

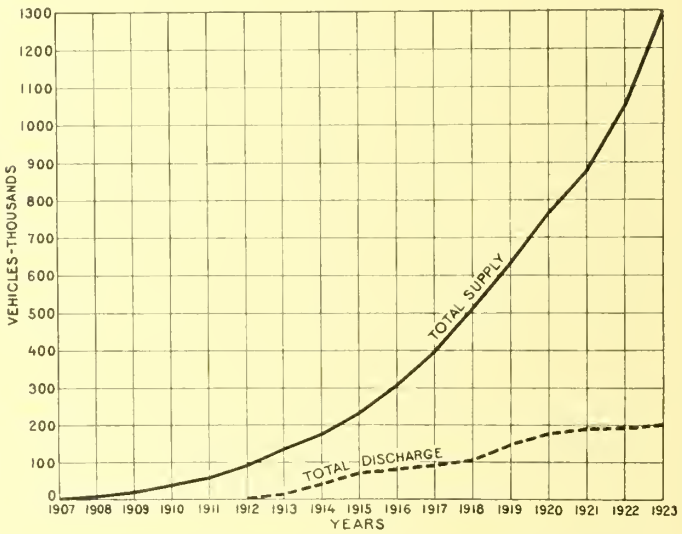


FIG. 5.—Supply and discharge curves for motor vehicles

The two highways selected for analysis are routes Nos. 2 and 4, running south from San Francisco and Sacramento, respectively, to San Diego and Los Angeles, with practically continuous hard surfacing, mostly of concrete and concrete with an asphaltic concrete top. The combined length of the two roads is approximately 857 miles, excluding mileage in towns and cities. The daily average traffic profiles, as derived from the 1920 and 1922 traffic counts, are shown for these two routes, respectively, in Figures 1 and 2. The costs of the original construction of these routes are known, as are also the costs of reconstruction and improvement and the approximate annual maintenance cost. The cost information available shows the annual cost per mile for the two routes to be approximately as follows: [9]

	Annual cost per mile
Bond requirements, 6 per cent on $19,270 construction cost per mile (exclusive of Federal aid)	$1,156
Maintenance and reconstruction	1,327
Improvement	1,101
Total annual cost per mile	3,584

[9] Construction costs, taken from the State records, include general items amounting to 14 per cent of the distributed items. Up to June 30, 1923, the cost had amounted to $17,616,088, from which is subtracted $1,101,566 of Federal-aid funds. The balance of $16,514,522 derived from bond revenues is equivalent to $19,270 per mile. Bond requirements of 6 per cent include approximately 1½ per cent for amortization requirements.

Of this total annual charge of $3,584 per mile, the bond requirements are already met by general State taxes. There remains a balance of $2,428 to be found from current revenue if this figure prevails for the next succeeding year. What portion of this balance which must be met with current revenues will be covered by the taxes on motor vehicles?

California taxes motor vehicles in two ways: (1) By a tax of 2 cents per gallon on gasoline; and (2) by a flat license fee of $3 for automobiles and a surcharge for trucks. The portion of the total taxes collected in this way which can properly be allotted to routes 2 and 4 can be determined in the following manner: From the 1922 traffic count it has been found that the total annual vehicular movement on all roads and streets is approximately 8,500,000,000 vehicle-miles. The daily movement on the two roads is 1,607,400 vehicle-miles. Allowing 8 per cent for night traffic as shown by the 1922 traffic count and remembering that the traffic for the first five days of the week is only about 58 per cent of the weekly total, the total annual traffic will be found to be approximately 784,000,000 vehicle-miles.

Comparing this measure of the traffic on the two roads with the total traffic of 8,500,000,000 vehicle-miles the ratio of 9.22 per cent is obtained, and this ratio may be applied to the total collection of license fees and truck surcharges to obtain the amount which may properly be allocated to the two roads in question. Assuming then an average motor-vehicle performance of 15 miles per gallon of gasoline, an average surcharge on the 43,000 motor trucks in the State of $25, and allowing 6 per cent for collection of license fees and assuming that there is no additional charge for collection of gasoline taxes, the following revenues which may be allocated to the two roads result:

Total gasoline tax on 784,000,000 vehicle-miles, at $0.001333		$1,045,072
9.22 per cent of $3 license fees on 1,100,000 vehicles	$304,260	
9.22 per cent of $25 surcharge fees on 43,000 solid-tired trucks	99,115	
	403,375	
Less 6 per cent for collection expenses	24,202	
		379,173
Total revenues, routes 2 and 4		1,424,245
Average per mile		1,662

This analysis of operating revenues and expenses is made from existing current data, but some adjustments are required. Since truck registrations and total vehicle registrations for 1924 are not known, the 1924 rates are applied to 1922–23 traffic figures. The total operating income per vehicle-mile on the 8.5 billion vehicle-miles annually, indicated for 1922–23, is $0.001847. This figure includes prorated gasoline tax, $0.001333; license fee, $0.000388; and prorated truck surcharge, $0.000126. The indicated gross revenue is thus about $15,700,000. This figure will doubtless be exceeded in 1924. Consequently the apparent operating deficit of $766 per mile from the first analysis is subject to decrease, unless the costs of maintenance and reconstruction and improvement correspondingly increase with the traffic.

Another analysis may now be made. Since the State highway commission by law receives one-half the net motor revenues for maintenance and improvement of State highways, the current State highway income from this source is about $7,500,000. This income divided by 1.5 billion vehicle-miles of indicated State highway operation yields $0.005 of operating revenue per vehicle-mile. To the 784,000,000 vehicle-miles produced on routes 2 and 4 there accrues then $3,920,000, or $4,574 per mile, or a surplus of $2,146 over the current requirements on this route. If the bond requirements were included in the charges the surplus would reduce to $990 per mile. Routes 2 and 4 are the heaviest traveled long routes in the State highway system and the operating revenue is higher in proportion to expense than the average for the entire system. A study of the entire system by the first method of analysis would reveal a larger proportionate deficit, and similarly by the second method probably a smaller surplus, than the figures given above for routes 2 and 4.

The foregoing analysis necessarily contains some tentative assumptions, particularly with respect to the vehicle-miles per gallon of gasoline and the percentage of total annual operation of vehicles on the combined State highways and county roads. The analysis is given, however, to indicate a method by which highway operating costs and highway revenue may be compared. Manifestly the difference between vehicle taxation and benefits to the vehicle by improved highways is still too wide to warrant over-refinement of the basic assumptions. Apparently the benefit of the highway to the vehicle is at least five times the cost of upkeep of the highway to the vehicle only, and this is probably the important element of modern highway construction.

There is perhaps no stronger reason for the continuation of State-wide traffic counts than the necessity for accurate measure of annual traffic as a basis for determining the correct taxation of motor vehicles. The division of motor-vehicle revenues between State and local purposes requires further investigation. The principle of paying for country roads by city revenue, which was begun with the advent of State aid in Massachusetts and New Jersey in the early nineties, is being carried out in California and other States in a new form under the present motor-vehicle taxation by reason of the evident large movement of traffic in cities from which revenue accrues to the State and county highways. If State highways are to be extended for the general benefit of the people, this principle of taxing the city traffic for country roads must continue.

HIGHWAY RESEARCH BOARD TO HAVE NEW CONTACTS

Appointment at an early date of representatives of various engineering schools to serve on the Highway Research Board was announced by Director Charles M. Upham at the recent annual meeting of the board at Washington, D. C. The appointment of the college representatives follows the policy of the new management of the board recently announced, which is to keep in close touch with the various agencies conducting highway research or employing its findings through the medium of "contact men." Similar appointments of representatives of the State highway departments were announced a short time ago.

THE ECONOMICAL USE OF WHEEL SCRAPERS

By J. L. HARRISON, Highway Engineer, United States Bureau of Public Roads

FOR many years the wheel scraper has been a standard implement for moving earth in grading operations. In the days of the slip scraper, the wheeler was used for the haul distances which were too long for economical slip work and too short for economical wagon work. More recently the fresno has supplanted the slip to a great extent, because it uses relatively more horsepower and relatively less man power. This has made it an economical implement for the short haul and has extended its effective range of haul as compared with that of the slip, thus raising, somewhat, the economical minimum haul distance for the wheeler as commonly employed.

Meanwhile, the elevator grader and the steam shovel, by shortening the time required in loading wagons, have lowered the minimum haul distance at which wagons and trucks are effective dirt movers, and thus have shortened the maximum haul required

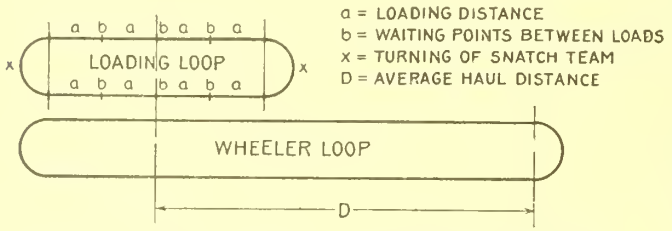


FIG. 1.—Diagram of wheeler and snatch team operation

of wheelers. As a result the haul distances for which the wheeler may be considered standard equipment have been somewhat modified.

Although there yet remains a field in which it is the most economical implement, its exclusive use under any but exceptional circumstances is precluded by the very fact that its most effective use is in hauls of moderate length. It is, rather, a tool to be used in connection with some other implement, and on highway work this is generally the fresno, although occasionally a slip-wheeler combination is encountered. From the practical standpoint, therefore, the operation of wheelers can not be dealt with as a problem by itself. Even the number of wheelers that are to be used on a job may be secondary to the question of how many fresnoes or how many slips it is best to use. In any event, as both fresnoes (or slips) and wheelers are likely to be used on any job where either is used, one of the problems that confronts the contractor at once is the determination of the proper point at which to shift from one to the other. On the basis of general theory this shift should be made at a little over 300 feet. This is probably accurate enough for ordinary estimating purposes. Moreover, it is quite as accurate as current contracting practice for contractors make the shift at from something under 300 feet to something over 400 feet without regard to theoretical considerations and often allow the maximum fresno haul to overlap the minimum wheeler haul, preferring to do this rather than bother with too frequent exchanges of equipment. This is natural enough when one considers that the change from fresnoes or slips to wheelers calls for the use of different methods in handling the job and means that extra men must be secured if the whole outfit of teams is to be employed. But while the proper exchange point as between fresnoes and wheelers is generally assumed to be under 350 feet, for outfits of ordinary size if all of the outfit is utilized a generalization of this kind overlooks two facts of considerable importance.

(1) An error of 50 feet in the selection of the exchange point will often affect the unit cost at which excavation is handled by 10 per cent or more.

(2) The size of the outfit together with the pay scale prevailing may affect the exchange point by more than 100 feet.

Therefore, on projects involving a large amount of haul between 275 feet and 450 feet the cost of material moved over these distances may easily be affected as much as 30 per cent and sometimes more by error in the selection of the exchange point. Such a difference may be sufficient to turn a possible profit into an actual loss.

SIGNIFICANT DIFFERENCES BETWEEN WHEELER AND FRESNO OUTFITS

Before attempting to show how such losses can be avoided it will be well to direct attention to certain well-known differences between normal fresno and wheeler outfits which have an important bearing on their relative production. The standard 4-foot fresno is ordinarily drawn and loaded by 3 mules, and the driver is able to load and dump without help. In a fresno outfit the only force required in addition to the drivers and teams for the fresnoes themselves is that which is needed for plowing. Each wheeler, on the other hand, requires 2 mules and a driver for the haul, but for loading and dumping additional force is required which usually consists of a snatch team, a loader, and a dumper. The force required for plowing is the same in either case. For example, a typical 24-mule outfit organized for fresno operation would employ 18 mules and 6 drivers for 6 fresnoes, 4 mules and 1 driver for the plow, leaving 2 extra mules. Organized as a wheeler outfit 16 mules would be hitched to wheelers with 8 drivers, 2 mules and 1 driver would be required for the snatch team, 4 mules and 1 driver for the plow, and there would be the same reserve of 2 mules, but 2 extra men would be required to load and dump. In other words, the wheeler outfit requires 5 more men than the fresno force for the same number of mules, and diverts 2 mules from hauling to loading.

WHEELER PRODUCTION LIMITED BY SNATCH-TEAM CAPACITY

These differences in organization between wheeler and fresno outfits have an important bearing on the relative production of the two kinds of equipment at various hauls and the cost of the production. Inherent in them are the factors which determine the haul distance at which it is profitable to substitute a wheeler outfit for a fresno outfit and vice versa. The fact that the wheeler outfit needs a snatch team which is not required in the fresno organization has a very important effect upon the relative output of the two organizations. In the case of the fresno, therefore, output depends directly on the haul distance. When

the haul distance is short the output is relatively large and vice versa, and there is a uniform decrease in output as the haul distance increases.[1] The output of the wheeler organization, on the other hand, is limited in the shorter hauls, not by the length of the haul, but by the capacity of the snatch team to load the wheelers as quickly as they return for loading. With increase of haul distance, however, a point is reached beyond which the output is governed, as in the case of fresnoes, by the time required for the wheeler haul.

The relation of the snatch team to the rest of the job is shown graphically in Figure 1. Actually the wheeler loop is superimposed on the loading or snatch-team loop. It is separated in the figure merely for convenience and clarity.

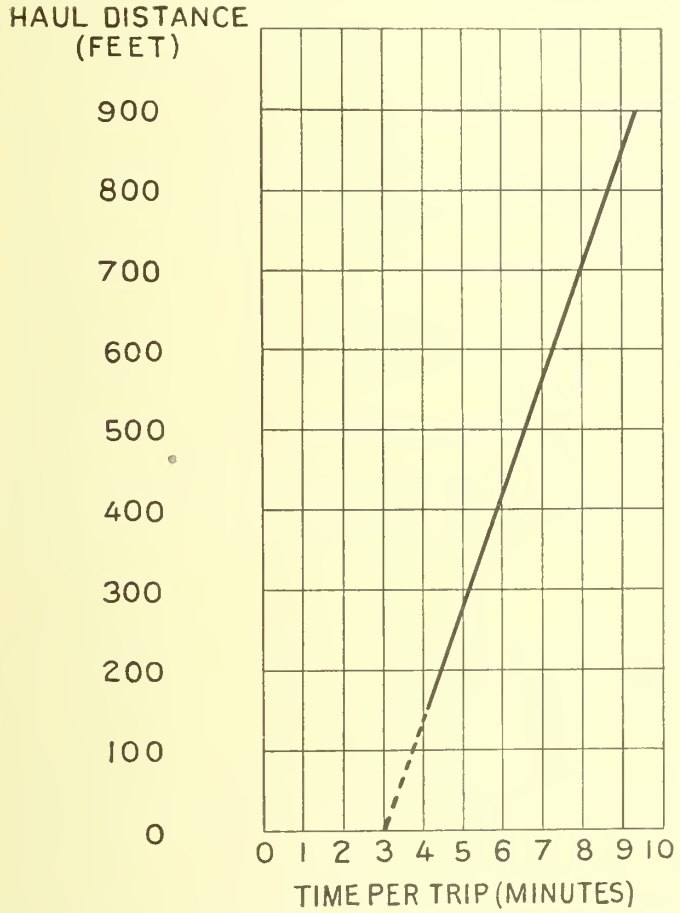


FIG. 2.—Relation between length of haul by No. 2 wheel scraper and time per trip

The loading loop consists of a series of loading operations, each extending over a distance a (fig. 1), followed by waiting periods at points b, during which the loaded wheeler is driven out and the next wheeler is driven in and coupled up. In practice the distance required in taking on a load a varies considerably, but averages from 12 to 15 feet. The time required to load is about 0.35 minute. The exchange of wheelers takes about 0.4 minute. The turns at the end of the loading loop (x in fig. 1) require about 0.7 minute, which in practice takes the place of a waiting period b, the slight difference in time required being so small as to be of no significance. Practically, then, the loading operation absorbs 0.75 minute. Knowing this, the number of wheelers that can be loaded with one snatch team without delay can be calculated from the time-distance graph (fig. 2) which gives the average time required in moving a load over ordinary working distances.[2] An illustration of its use for this purpose follows:

Suppose the average haul distance D (fig. 1) is found to be 400 feet. By reference to the time-distance graph (fig. 2) it will be seen that the average time per round-trip for a 400-foot wheeler haul is 5.9 minutes. As the average time required to load one wheeler is 0.75 minute, it follows that 8 wheelers (5.9÷0.75) operated with average efficiency will deliver a wheeler supply to the snatch team which will slightly exceed the capacity of the snatch team as commonly managed. By substituting the appropriate figures for loading time and trip time the same method can be used to determine the economical number of wheelers for any haul distance and for any job. On a well-managed job the average loading time of 0.75 minute will be reduced; a poorly handled outfit will require a longer time. For ready reference the following table, based on average efficiency in operating both the snatch team and the wheelers. is included:

TABLE 1.—*Number of wheelers which can be loaded by one snatch team at various haul distances with average management*

Length of haul (feet)	Number of wheelers for each snatch team
165 to 270	6
270 to 375	7
375 to 480	8
480 to 585	9
585 to 690	10
690 to 795	11
795 to 900	12
900 to 1,005	13
1,005 to 1,110	14

TIME OF LOADING NOT AFFECTED BY KIND OF MATERIAL

Within the range of materials observed in the bureau's studies which have been confined to the Mississippi Valley, the kind of material, whether sand, light gravel, loam, or clay, appears to have comparatively little influence on the time used in loading. Plowing any of these materials is a matter of power. Heavy materials will require an extra team on the plow and an extra horse on the snatch team. When these are provided the time required for loading the wheelers can be kept reasonably constant, regardless of the material handled.

So long as the snatch team is unable to load all of the wheelers which can be made available the wheeler output is practically constant and is fixed by the rate at which the snatch team can work. The output remains constant until the haul distance becomes so long that the full supply of wheelers is thrown in, after which any further increase in haul distance is accompanied by a drop in production. Over what range of haul distance the output will remain constant will depend on how many wheelers are available. The bearing of this fact on the output of a wheeler organization

[1] See PUBLIC ROADS, Vol. 5, No. 8, October, 1924. The Cost of Grading with Fresnoes, p. 10.

[2] For the derivation of the time-distance graph and its significance see PUBLIC ROADS, Vol. 5, No. 7, September, 1924. The Effect of Haul on the Cost of Earthwork, p. 14.

and its effect on the relative cost of wheeler and fresno operation are shown in graph B, Figure 3, from which it is clear that when this condition prevails the exchange point may be such as to involve the use of a fresno haul somewhat longer than is commonly supposed to be appropriate.

The contractor can sometimes obtain an advantage by working surplus stock on fresnoes. The result of this, if the fresnoes are worked in parallel with wheelers, is to increase output (see graph C, fig. 3) with relatively little increase in cost of operation, so that when this is done it may become appropriate to exchange fresnoes for wheelers at a somewhat reduced haul distance. Each case, however, should receive independent study, and any effort to treat it in any other way would lead to erroneous impressions.

Part of a typical series of stop-watch readings on snatch team operation in sandy loam soil is presented

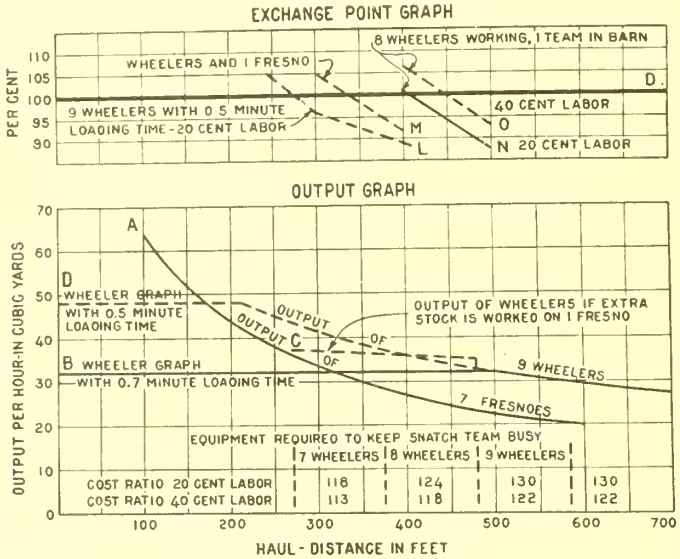

EXCHANGE POINT GRAPH
8 WHEELERS WORKING, 1 TEAM IN BARN
WHEELERS AND 1 FRESNO
9 WHEELERS WITH 0.5 MINUTE LOADING TIME-20 CENT LABOR
40 CENT LABOR
20 CENT LABOR
PER CENT
OUTPUT GRAPH
WHEELER GRAPH WITH 0.5 MINUTE LOADING TIME
B WHEELER GRAPH WITH 0.7 MINUTE LOADING TIME
OUTPUT OF WHEELERS IF EXTRA STOCK IS WORKED ON 1 FRESNO
9 WHEELERS
7 FRESNOES
EQUIPMENT REQUIRED TO KEEP SNATCH TEAM BUSY
7 WHEELERS
8 WHEELERS
9 WHEELERS
COST RATIO 20 CENT LABOR
COST RATIO 40 CENT LABOR
OUTPUT PER HOUR-IN CUBIC YARDS
HAUL-DISTANCE IN FEET

FIG. 3.—Output and exchange point graphs for a small-sized outfit. An outfit of this size is used to illustrate the fact that some outfits are not well suited to handle work involving hauls of from 300 to 600 feet

in Table 2. The fact that the loading time varies from 12 to 33 seconds and the waiting interval between loads from 14 to 37 seconds suggests that with a little more attention to detail the output of the snatch team could be increased from a load every 46 seconds (0.75 min.) to a load every 30 or 36 seconds (0.5 to 0.6 min.). The result of this would be to make it possible to utilize all of the stock on the wheelers at haul distances much lower than is now generally practical. The effect of such a reduction in loading and waiting time on the output at the shorter hauls is shown by dashed lines in Figures 3 and 4. In the case illustrated in Figure 4 this would be of no advantage, but in the case illustrated in Figure 3 the advantage would be considerable. The cost of fresno yardage at 275 feet, the exchange point if the wheeler loading and waiting time can be reduced to 0.5 minute (see exchange point graph, fig. 3[3]), as compared with the cost of yardage moved by fresnoes 405 feet, the normal exchange point (with 20-cent labor), is about in the ratio of 100 to 135. At 405 feet wheeler operation with 0.5 minute loading time costs about 88.5 per cent of the cost of fresno operation at the same haul distance. The saving effected by the use of wheelers with this reduced loading time would therefore amount to about 11.5 per cent at 405 feet and would average about 6 per cent on earth moved between 275 and 405 feet.

TABLE 2.—*Stop-watch readings on snatch-team operation in sandy-loam soil*

Loading time	Waiting interval between loads	Waiting interval due to lack of wheelers	Loading time	Waiting interval between loads	Waiting interval due to lack of wheelers
Seconds	*Seconds*	*Min. Sec.*	*Seconds*	*Seconds*	*Min. Sec.*
12	19	- --	15	--	2 7
23	19	- --	19	--	0 51
16	22	- --	18	--	2 55
15	--	3 50	22	25	- --
19	24	- --	20	30	- --
28	26	- --	20	--	1 48
33	18	- --	20	28	- --
25	21	- --	17	--	2 3
24	--	2 1	25	30	- --
22	35	- --	18	30	- --
15	--	1 2	19	--	2 0
15	--	1 43	35	--	1 25
25	--	2 54	29	34	- --
16	20	- --	33	17	- --
17	27	- --	20	37	- --
17	23	- --	27	--	2 55
13	--	1 8	32	--	1 8
16	--	2 30	23	--	1 38
24	22	- --	25	--	0 52
25	20	- --	25	--	2 6
18	--	0 55	30	--	0 47
17	21	- --	33	--	1 0
18	--	1 23	18	--	0 54
17	14	- --	25	--	0 45
15	18	- --	25	--	1 15
19	--	1 7	26	--	0 59
14	18	- --	24	34	- --
20	--	1 20	31	20	- --
14	19	- --	16	--	0 45
19	24	- --	24	25	- --
19	--	0 43	19	--	1 45
25	22	- --	30	34	- --
--	19	- --	19	20	- --
21	33	- --	33	--	- --
20	21	- --			

Average loading time	seconds	22
Average waiting interval between loads	do	24
Average distance traveled in loading	feet	13

Operating in this fashion would require constant attention, but it is only by giving attention to details of this sort that satisfactory profits can be secured. The snatch team is the key of a wheeler job whenever there is a surplus of wheelers because its production sets an absolute limit on output no matter how many wheelers may be made available. It is so futile to attempt to boost production by adding more wheelers after the capacity of the snatch team has been reached that it seems trite to lay emphasis on it, but the tendency to make this effort has been so apparent on many of the jobs which have been observed that an impression has been formed that contractors often overlook small repetitive delays, forgetting that in the course of a day these may cost quite as much as it would cost to have a team or two with their drivers standing idle at the side of the road. Generally there is miscellaneous work which extra teams can do, and much would be saved if teams which can not be effectively used in moving earth were employed on work of this sort.

OPERATING COST RATIO AN IMPORTANT FACTOR IN COMPARING FRESNO AND WHEELER OPERATION

In the foregoing, wheeler and fresno outfits have been compared on the basis of their respective outputs,

[3] The exchange point graph is derived from the output graph by dividing the output of the fresno force for any haul distance by the output of the wheeler force at the same haul distance and multiplying the result by the operating cost ratio, thereby obtaining the relative efficiency of wheeler movement from the standpoint of cost per cubic yard with respect to the unit cost of fresno operation considered as 100 per cent. The manner of plotting the exchange point graph is described in detail on page 19.

and it has been shown that at hauls ranging from 300 to 500 feet the wheeler output becomes greater than that of the fresno force. This larger output is apt to be accepted as evidence of superior performance without giving adequate attention to the difference in the cost at which the wheeler yardage is secured. To avoid reaching erroneous conclusions every contractor should have clearly in mind, as an important item in determining when to change equipment, an accurate statement of the relative cost of operating as a fresno outfit and as a wheeler outfit. An estimate, developed to show the nature of the operating cost ratio follows:

Comparative estimates of the cost of working the same outfit on fresnoes and on wheelers

	With teamsters at 20 cents an hour		With teamsters at 40 cents an hour	
	6 fresnoes	8 wheelers	6 fresnoes	8 wheelers
Outfit:				
4 mules on plow, 18 mules on fresnoes, 2 mules extra; 24 mules, total	[1] $15.00	[1] $15.00	[1] $15.00	[1] $15.00
Add one-fifth for Sundays and lost time	3.00	3.00	3.00	3.00
1 foreman	4.00	4.00	6.00	6.00
1 plowholder	2.50	2.50	4.50	4.50
1 four-line driver	2.50	2.50	4.50	4.50
6 fresno drivers	12.00		24.00	
8 wheeler drivers		16.00		32.00
1 snatch-team driver		2.00		4.00
1 loader		2.50		4.50
1 dump man		2.00		4.00
1 extra laborer	2.00	2.00	4.00	4.00
Total cost	41.00	51.50	61.00	81.50
Operating cost ratio	100 to 126		100 to 133	
Less 1 driver if 1 wheeler is not used		2.00		4.00
Total cost	41.00	49.50	61.00	77.50
Operating cost ratio	100 to 121		100 to 127	
Less 2 drivers if 2 wheelers are not used		4.00		8.00
Total cost	41.00	47.50	61.00	73.50
Operating cost ratio	100 to 116		100 to 120	

[1] Cost used includes barn help, veterinary services, etc., as well as feed.

The cost estimate shows that to make wheeler operation in this case advisable as compared with fresno operation, all wheelers being in use, something more than 126 cubic yards must be moved by the wheeler organization for each 100 yards produced by fresno operation when the common labor wage is 20 cents per hour, and that something more than 133 cubic yards must be produced by the wheelers for every 100 yards which can be produced by the fresnoes when the common labor wage is 40 cents per hour. On the other hand, if less than the full number of wheelers are used the operating ratio drops (assuming, of course, that labor not used is not paid for). As it has been previously shown that on hauls under 375 feet the full equipment of an outfit capable of placing 8 wheelers and a snatch team on the job can not be utilized, the advantage of keeping extra teams at the barn as compared with sending them on to the job (unless they are put on fresnoes or set at miscellaneous necessary work) is apparent from the manner in which the operating ratio falls as teams are laid off, in this case dropping from 126 to 116 (with 20-cent labor), or about 8 per cent. Of course, if all of the wheelers could be loaded, the reduction in operating cost would be attended by an even greater reduction in the production and would result in a net loss. Thus (see fig. 3), if while the haul is in the neighborhood of 300 feet, wheelers being in use, two of the nine teams used in this illustration are kept in the barn, there is no loss in output because the snatch team can only load seven wheelers. Under such circumstances the reduction in the operating ratio, which is due to the employment of two less drivers, is a positive saving. But if two teams are laid up when the haul is 700 feet, though the operating cost ratio would be reduced as before (from 122 to 113—if the wage of labor is 20 cents), because the output would fall off 22 per cent, the increase in cost per cubic yard, as compared with production with a full force at work, would be about 18 per cent.

In this connection it might be noted that contractors operating wheelers have been found by observation to be rather careless about keeping available stock and equipment at work. There are, of course, many angles to this question, but in allowing stock to stand idle the ordinary contractor appears to be under the impression that his loss is only the feed consumed. As a matter of fact the increase in the cost per yard of output which is due to the fact that the cost of superintendence, plow team, snatch team, loader, dump man, etc., must be distributed over a smaller yardage is often a more important consideration than the feed involved.

There are certain refinements covering profit, depreciation, etc., which might be considered in drawing such an estimate as the one shown above. The contractor who desires to go into this matter in greater detail will include such items, but for governing field operations it is thought that the more simple form of estimate is preferable.

OPERATING COST RATIO AFFECTED BY WAGES AND SIZE OF OUTFIT

The above estimate is presented for purposes of illustration. It serves, however, to show that, depending on the wage scale prevailing, the ratio of the output of fresnoes to the output of wheelers for equal cost per yard on equal haul distances will vary considerably. If estimates such as the above are prepared for outfits of different size, it will be found that the size also affects the ratio. This is because of the relation of the cost of the snatch team or teams, the superintendence, the plow team, etc., to the output produced. Contractors should make up their own estimates, using prices actually paid, and develop their own operating cost ratios. No two jobs are exactly alike. The illustrations which have been given show the salient characteristics of operating cost ratios, but they are intended only as a guide to contractors in preparing ratios which take into consideration the special features of their own systems of management and practices in arrangement of organization and, more particularly, the special conditions governing the pay scale prevailing.

The next step in determining the point at which fresnoes should be exchanged for wheelers involves a knowledge of output. For average conditions output can be predicted from the time-distance graph. It is developed in detail in Figure 5 for one wheeler and one fresno. Operation which yields more than the quantities here shown is above average for highway construction, and operation which yields less than the quantities here shown is below average.

Having the above-mentioned points in mind, it is possible to determine the haul distance at which the wheelers should be placed in service, the number of

wheelers and the number of fresnoes which are to be operated being known. To do this proceed as follows:

(*a*) Plot output graph (see curve A, fig. 4) for the number of fresnoes which are to be used. If the contractor has collected the data on the efficiency of his outfit, he should use this, particularly if his outfit is

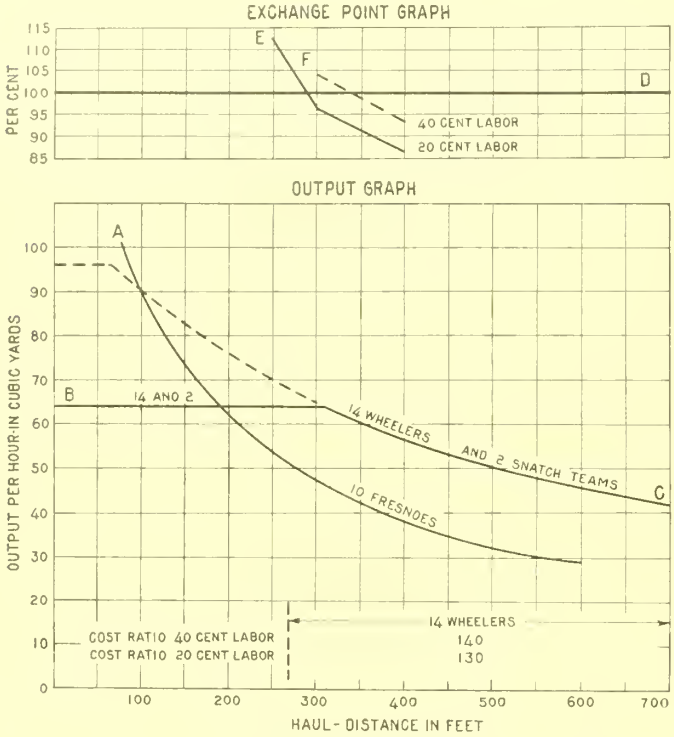


FIG. 4.—Output and exchange point graphs for medium-sized outfit. An outfit of this size is well suited to work involving hauls of from 300 to 600 feet. A large part of the yardage on most grading projects falls within these haul limits

above or below average in efficiency. If he has collected no data on his own outfit, the fresno graph given in Figure 5, which is for one fresno, should be used, the readings for every 50 feet being multiplied by the number of fresnoes which are to be used, and a new graph plotted from the products so secured.

(*b*) Draw a horizontal line (see B, fig. 4) at 32 cubic yards if one snatch team is used (at 64 cubic yards if two snatch teams are to be used).

(*c*) Beginning at the right at about 1,000 feet, plot the output graph for the number of wheelers that are to be used (see curve C, fig. 4) obtaining values from Figure 5 and proceeding as above) to an intersection with the horizontal line above referred to. The full wheeler output graph is that part of the horizontal line lying to the left of the intersection and the plotted graph lying to the right of the intersection.

(*d*) Proceeding with the development of the point at which an exchange between fresnoes and wheelers should be made, draw a horizontal line (see D, fig. 4) to represent the cost of fresno output at any haul distance, considered as 100 per cent.

	Fresnoes	Wheelers
Then, beginning at the right at 400 feet read output in cubic yard for fresnoes and wheelers—in this case	38	56.8
Divide wheeler yardage by operating ratio—in this case 1.30 (20-cent labor)	----	43.7

Divide fresno yardage by adjusted wheeler yardage as secured above: 38÷43.7=87 per cent.

Proceed in similar fashion at 300 feet and 200 feet, obtaining 97 and 127 per cent, respectively. These points when plotted and connected give graph E, Figure 4, which indicates that fresnoes should be exchanged for wheelers when the haul reaches 290 feet.

Calculated in a similar manner the dashed line F (fig. 4) gives the point of intersection, i. e., the point at which a transfer from fresnoes to wheelers should be made, when the wage of labor is 40 cents an hour—in this case 340 feet.

THE EFFECT OF MINOR DELAYS ON OUTPUT

Another matter to which the contractor should give careful attention is the operation of the wheelers. Figure 6 shows the relation of trip time to haul distance for a typical wheeler job as plotted from extended field observations. It will be observed that on this job, the results secured were slightly better than the average indicated by Figure 2, the constant or x intercept being 2.9 minutes per load instead of 3 minutes. Of this, only 1.25 minutes can be accounted for in such operations as turning at the cut (18 seconds), turning at the fill (16 seconds), manipulating pan (16 seconds), loading (21 seconds), and dumping (4 seconds). These are operations which must be performed with every load, and the fact that they sum only 1.25 minutes as against an average time consumption of 3 minutes per load, in addition to the time used in hauling and in returning from the dump draws attention to the importance of miscellaneous losses as a factor in cutting down output. This loss of 1.75 minutes per load is about 30 per cent of the time required for a 400-foot haul and about 20 per cent of the time required for an 800-foot haul. Such losses arise from all sorts of minor delays which, in their general nature, are similar to the losses encountered on fresno jobs.[4] These losses are prevent-

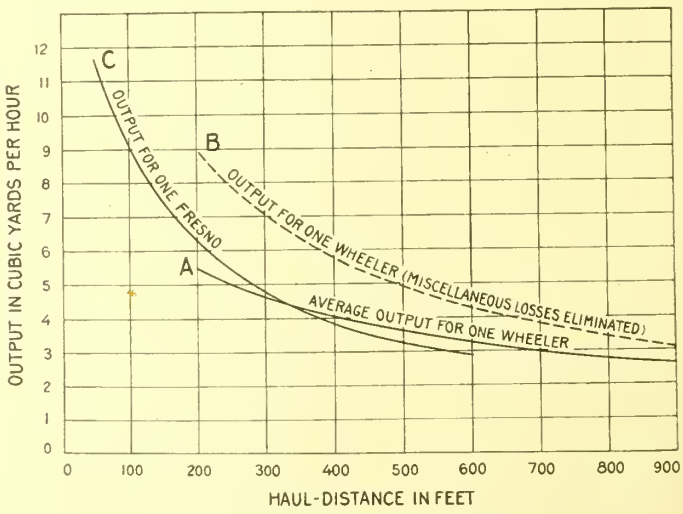


FIG. 5.—Average output curves for one wheeler and one fresno

able in a large measure, but only by persistent effort on the part of the foreman. A wheeler job is like any other construction operation in that it will not run itself. It must be carefully and continually directed. The tendency of grading foremen to allow their jobs

[4] See PUBLIC ROADS, Vol. 5, No. 8, October, 1924. The Cost of Grading with Fresnoes, p. 10.

to run themselves can be emphasized in no better way than by calling attention to the fact that these miscellaneous delays average more than 4 minutes for each cubic yard moved. The solid line A in Figure 5 shows what has been found by observation to be the average hourly output per wheeler. The dashed line B shows what it would be if these miscellaneous losses were totally eliminated. The value of real management on wheeler work should be apparent from these graphs.

Particularly in the case of large contractors who must work through field superintendents, the problem has always been to find some effective means of checking up on the efficiency of the grading work without being constantly on the job. This is done on other forms of construction work by first determining the force which will be employed and then determining how long it will take such a force to do the work. In the highway grading field it has not been possible to employ a similar method because no adequate data have been available from which to calculate the relation of haul distance to the time required to perform specific grading operations. This deficiency the time-distance graph supplies, and from it it is possible by simple methods to calculate the time required to perform any grading operation involving the use of wheelers, and thus schedule the job quite as accurately as any other contract operation can be scheduled. Once this is done, a contractor can handle distant work with confidence by simply keeping it up to the time schedule. Cost and profit having been determined in advance, his problem is reduced to one of finding a superintendent who can, with the force specified, keep within the time limit imposed on him. Moreover, as the contractor may have confidence that the time limit is reasonable, he can place his finger on the exact cause of failure to meet the time schedule whenever the job lags.

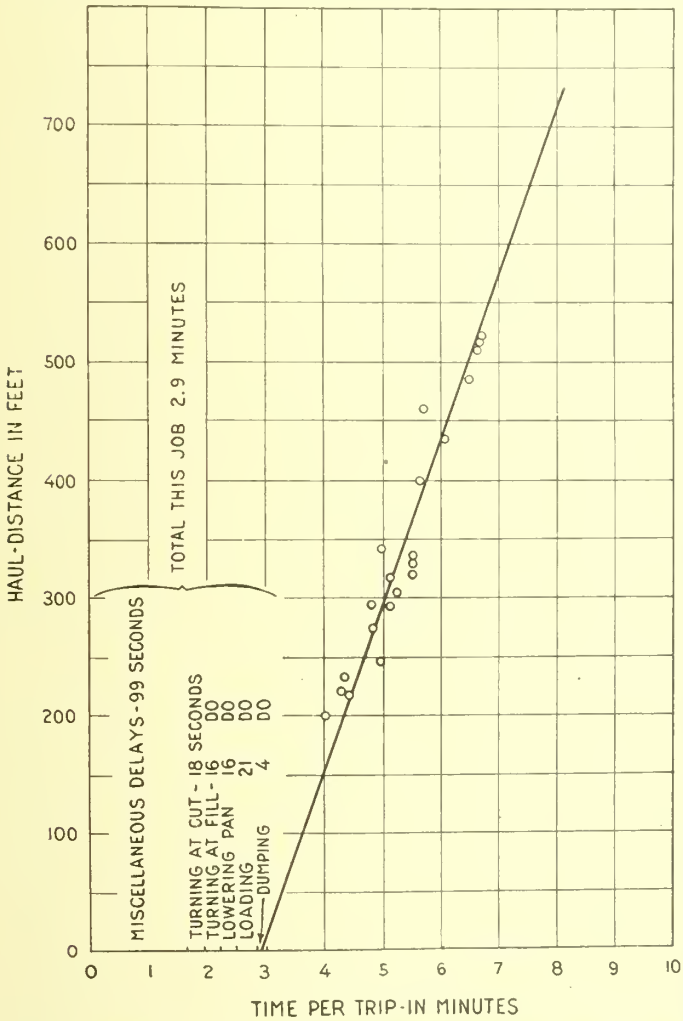


FIG. 6.—Relation of trip time to haul distance, showing various losses

TIME SCHEDULES SHOULD BE BASED ON PREVAILING CONDITIONS

Before entering into the details of the method of preparing a schedule, a number of matters should be briefly touched upon:

(1) The time-distance graph referred to above is an average-performance graph. The pitch of the time-distance line varies only slightly from job to job, indicating that mules walk at a fairly steady pace. On one or two horse jobs it has been found that there is some tendency for the animals to walk slower than this line indicates as standard. This suggests that horse jobs may require a little more attention than mule jobs, if the rate of travel is to be kept up to standard. On the whole, however, there appears to be less danger that losses will occur here than in the constant which, as shown, is known to contain about 1.75 minutes of miscellaneous avoidable loss per load moved. It, therefore, seems pertinent to note that while it would be improper to suggest the use of a constant of less than three minutes in estimating wheeler work until a contractor has developed foremen who can conduct the work with losses that are less than those now included in the constant, the wide-awake contractor will make an effort to eliminate such losses. Once he is sure that he has developed foremen who can work with lower losses he will adjust his formula for estimating accordingly. To do this all that is necessary is to reduce the constant, given in the estimating procedure below as 3, to whatever his experience shows he can attain.

(2) In estimating the cost of moving such materials as heavy clay, gumbo, etc., the correct procedure is to allow for the cost of extra power where it will be needed—on the plow and on the snatch team. Once the power is available at these two points, the rate of output of the wheelers can be kept practically as high as in lighter materials. As a matter of fact, the output often falls off when heavy materials are encountered, but this appears, generally, to be due to a failure to apply sufficient power at these two critical points.

(3) In preparing the time schedule which follows it has been assumed that Sundays and rainy days will absorb one-fifth of the time. Such an assumption is necessarily made for purposes of illustration only. The careful contractor will consult the Weather Bureau records for the locality where the work is to be done and, depending on the months during which he expects to operate, will determine the amount of wet weather for which he should allow. Obviously, it would be improper to make the same allowance for bad weather on an east Texas project to be done during the first quarter of the year and a central Dakota project to be executed in the third quarter.

(4) Another matter which deserves a word of comment is that the method here presented presumes the existence of data on which a contractor can determine for any given project what is wheeler work and what is fresno work. This is essential, as in almost every case where one of these implements is used the other must also be used. In some sections the wheelers are used with slip scrapers, but the latter are so inefficient (under Mississippi Valley conditions), as compared with the fresnoes, that they have practically disappeared. To draw up a proper cost estimate it is, therefore, essential that the contractor know not only the quantities to be moved, with their haul distances, but that he know the quantities, their location, and their haul distances for all cuts and borrow pits which involve a wheeler haul.

Generally the responsible engineers will gladly furnish such data to an interested contractor, but there are now a number of States where the information can be had only by special request.

HOW TO SCHEDULE A WHEELER JOB

Assuming that the basic data are available, the following method may be used in calculating the time required in executing a wheeler project:

(1) Multiply the wheeler yardage of each cut or borrow pit by the sum of 3 plus 7/10 of the distance in stations over which it must be moved. As the latter factor represents the time in minutes required to move a wheeler load, which is 4/10 of a cubic foot, the product is 4/10 of the time in minutes required to move the given yardage.

(2) Divide the product for each cut, as obtained above, by 240 to obtain the number of 10-hour work days required to move the yardage with one wheeler. (Use 216 for a 9-hour day and 192 for an 8-hour day. These factors are 4/10 of the number of minutes in the working day.)

(3) Divide the number of work days for one wheeler, as obtained above for each cut, by the number of wheelers that can be used with one snatch team, as shown in Table I, to obtain the number of work days per outfit in each cut (or borrow pit). If the number of wheelers that will be available is less than the number which can be used with one snatch team—a condition likely to occur where small outfits are engaged on long hauls—divide by the number of wheelers which will be available. If the outfit is so large that two snatch teams must be used on the shorter hauls, divide by twice the number of wheelers called for in Table I or by the total number of wheelers which can be supplied, using whichever of these gives the smaller number.

(4) The sum of the "day's work for the outfit" obtained for the various cuts (and borrow pits) is the number of day's work required for the job. This presumes that one outfit will be placed on the job. If two or more outfits are to be used, the sum developed should be appropriately divided by the number of outfits that will be worked.

A brief illustration follows. In working up this time schedule it has been developed for two outfits; the first (outfit A), 6 fresnoes or 8 wheelers, and the second (outfit B), 10 fresnoes or 14 wheelers.

Quantities and haul distances

	Fresno operation		Wheeler operation	
	Quantity	Average haul	Quantity	Average haul
	Cu. yds.	*Stations*	*Cu. yds.*	*Stations*
Mile 1 (cuts)	1,100	2.25	820	6.82
	820	1.80	373	3.80
	778	1.50		
	550	1.90	352	4.90
	722	2.20	1,320	8.62
	430	2.30	280	5.97
Total	4,400		3,145	
Mile 2 (cuts)	875	2.25	1,020	7.35
	490	1.75		
	550	2.25	792	6.06
	635	2.15	530	5.77
	580	2.10	513	5.29
	420	2.00	360	3.51
	625	2.20	745	5.88
	845	2.30	865	8.62
Total	5,020		4,825	

Detailed time schedule

	Day's work for 1 fresno	Day's work for outfit A	Day's work for outfit B	Progress schedule	
				Outfit A	Outfit B
Mile 1 (Fresno work):[1]					
4,400×1.2 =5,280				Start May 1	Start May 1.
1,100×2.25=2,475					
820×1.8 =1,476					
778×1.5 =1,167					
550×1.9 =1,045					
722×2.2 =1,588					
430×2.3 = 989					
200) 14,020	70.1	11.7	7.0		
Mile 1 (Wheeler work):					
820/240 (3+0.7×6.82)	26.6	3.33	1.90		
373/240 (3+0.7×3.80)	8.8	1.10	0.63		
352/240 (3+0.7×4.90)	9.4	1.18	0.67	Allow ½ day for bad weather.	
1320/240 (3+0.7×8.62)	49.7	6.21	3.55		
280/240 (3+0.7×5.97)	8.4	1.05	.60	Finish May 29	Finish May 17 (noon).
		12.87	7.35		
Mile 2 (Fresno work):					
5,020×1.2=6,024					
875×2.25=1,969					
490×1.75= 858					
550×2.25=1,238					
635×2.15=1.365					
580×2.10=1,218					
420×2.00= 840					
625×2.20=1,375					
845×2.30=1,944					
200) 16,831	84.2	14.0	8.4		
Mile 2 (Wheeler work):					
1020/240 (3+0.7×7.35)	34.6	4.32	2.47		
792/240 (3+0.7×6.06)	23.9	2.99	1.71		
530/240 (3+0.7×5.77)	15.5	1.94	1.11	Allow 1 day for bad weather.	Allow ½ day for bad weather.
513/240 (3+0.7×5.29)	14.3	1.79	1.02		
360/240 (3+0.7×3.51)	8.2	1.17	.59		
745/240 (3+0.7×5.88)	22.1	2.76	1.58		
865/240 (3+0.7×8.62)	32.6	4.07	2.33	Finish July 10	Finish June 11 (noon).
		19.04	10.81		

[1] For method of scheduling fresno work see PUBLIC ROADS, Vol. 5, No. 8, October, 1924. The Cost of Grading with Fresnoes, p. 10.

The above time schedule shows that outfit A (working 6 fresnoes or 8 wheelers) in handling this work will have to operate 25.7 days as a fresno outfit, and 31.91 days as a wheeler outfit. Similarly outfit B (working 10 fresnoes or 14 wheelers) will have to operate 15.4 days as a fresno outfit and 18.16 days as a wheeler outfit. To reduce these data to a cost estimate it is only necessary to multiply the number of fresno days by the cost of a day's work calculated as shown above in estimating the operating ratio, and to add to this the cost of the wheeler work obtained in the same way. To this must be added such items as the cost of getting to and away from the job, depreciation, wintering stock, as well as cost of complying with specifications as to maintaining completed grade, etc. Finally the profit should be added. The sum of these items divided by the number of cubic yards to be handled will give the appropriate price to bid on the job.[5]

[5] For a more detailed discussion of the method of estimating, see PUBLIC ROADS, Vol. 5, No. 8, October, 1924. The Cost of Grading with Fresnoes, p. 10.

An estimate developed in this way includes overhaul in the bid price.

NORTH CAROLINA INVESTIGATING PRESSURE ON PIPE CULVERTS

The vertical pressure transmitted to pipe culverts through earth fills is the subject of an experimental investigation which has been under way for some time by the North Carolina State Highway Department in cooperation with the University of North Carolina.

The experiments are being conducted with full-sized pipes, which are covered with fills of various materials to depths ranging up to 20 feet. The earth fill rests on the upper semicircumference of the pipe. Two concrete slabs in the plane of the horizontal diameter support the soil at the sides of the pipe and allow the space beneath the pipe to remain open for the weighing apparatus, which consists of four Riehle testing machines.

The investigation is being carried on at Chapel Hill, N. C., under the direction of Dean G. M. Braune.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

NOVEMBER 30, 1924

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 766,721.98	$ 380,509.23	85.3	$ 15,333,651.52	$ 7,436,225.92	838.0	$ 79,416.15	$ 39,708.07	0.6	$ 1,210,272.24	Alabama
Arizona	8,338,365.41	4,287,683.88	527.8	713,173.87	406,825.70	37.5	1,758,155.58	1,065,094.20	150.4	504,350.11	308,208.92	55.2	1,427,888.30	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	1,537,568.79	688,383.74	68.1	6,453,537.33	2,651,817.83	339.6	642,304.57	289,362.76	53.7	1,008,490.04	Arkansas
California	12,999,075.03	5,647,148.17	533.7	3,373,385.31	1,799,005.25	135.5	12,121,739.55	6,306,673.24	394.2	505,746.86	298,390.64	8.2	3,042,088.70	California
Colorado	8,108,070.31	4,029,898.97	502.6	880,626.15	444,393.39	47.6	5,316,483.73	2,831,641.04	199.9	35,740.10	20,057.34	2.9	2,173,890.26	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	199,024.56	98,423.00	4.9	3,262,054.48	1,045,804.04	54.1				967,409.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	462,969.09	197,825.82	13.8	1,318,788.24	504,231.10	33.1				29,758.25	Delaware
Florida	961,134.07	461,470.92	48.8	1,415,010.74	681,331.64	25.4	8,274,647.58	4,060,548.44	236.4	398,642.71	171,123.94	10.2	912,412.06	Florida
Georgia	17,167,373.32	7,955,805.20	1214.2	1,245,033.93	597,200.65	113.1	11,053,717.87	5,271,180.34	782.7	754,833.02	346,867.14	38.6	278,843.67	Georgia
Idaho	8,181,697.92	4,092,395.52	506.8	243,867.57	139,962.62	18.7	2,531,548.92	1,519,975.55	170.5	430,193.23	283,175.76	31.3	648,202.55	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	1,355,788.15	670,702.08	40.5	15,741,331.50	7,820,073.10	538.7	53,063.82	26,531.91	2.0	2,639,638.58	Illinois
Indiana	7,577,444.76	3,655,540.97	225.7	1,572,755.01	773,197.42	56.6	15,763,091.38	7,675,466.21	502.3				2,208,187.40	Indiana
Iowa	23,195,778.19	9,237,031.86	1682.9	837,879.52	354,112.26	74.1	8,319,885.79	3,840,599.47	585.4	1,364,138.90	613,800.00	59.7	1,290,593.41	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	4,996,700.82	1,901,478.91	146.0	14,944,690.02	6,008,613.63	630.5	2,895,451.26	1,338,396.49	158.1	7,623.17	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	1,755,750.19	717,266.65	63.7	8,336,630.31	3,828,889.32	321.4	474,276.14	204,477.96	17.1	1,007,157.79	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	1,358,277.16	662,014.97	127.0	4,808,164.95	2,389,108.72	284.1	191,242.29	95,621.14	1.1	482,553.81	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	295,533.42	147,087.77	12.3	1,704,509.38	823,525.56	62.4	113,667.57	56,833.78	6.3	762,589.51	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	358,411.02	179,205.49	16.4	2,801,989.31	1,199,249.89	85.3	171,075.41	52,750.00	4.2	4,422.84	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	88,434.91	28,760.00	1.4	6,555,098.99	2,103,931.92	107.3	466,585.01	137,365.00	7.1	1,543,995.86	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5	300,204.10	144,889.87	15.1	16,355,771.29	7,750,146.06	551.3				1,924,123.84	Michigan
Minnesota	24,037,561.24	9,885,843.07	2292.0	2,615,732.90	1,105,086.98	175.3	9,515,463.39	4,218,900.00	849.0	1,051,402.19	81,300.00	117.2	27,288.95	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	765,843.84	381,194.15	50.7	8,531,159.03	4,251,388.50	518.4	670,401.44	338,200.70	53.9	734,548.26	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.5	1,317,280.96	650,328.61	100.6	21,821,865.23	9,892,913.18	852.4	2,838,049.81	898,120.55	109.5	1,252,926.48	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	693,306.93	462,086.35	77.2	2,421,045.82	1,804,547.70	213.3	1,047,664.72	595,727.63	118.4	3,719,720.20	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	375,488.75	175,160.67	26.9	7,884,545.40	3,859,752.19	831.7	916,341.27	458,170.60	109.1	3,243,170.95	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	1,102,421.45	928,932.16	92.2	4,477,532.42	3,774,003.69	383.1	107,368.80	93,647.06	16.0	240,113.11	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	126,997.20	61,883.10	5.1	1,645,336.04	774,553.81	53.8	29,611.26	13,680.00	0.9	96,979.51	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	580,788.45	210,700.00	10.5	10,201,407.15	2,301,941.24	79.2				815,074.27	New Jersey
New Mexico	5,306,286.45	2,758,849.68	714.3	814,405.30	476,119.86	77.2	6,719,429.09	4,452,053.64	673.6	250,983.39	154,354.77	16.7	747,954.05	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	1,037,210.72	427,221.06	33.4	30,206,722.96	11,457,356.72	674.7	5,778,138.00	1,579,790.00	102.8	4,985,935.78	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	2,948,359.40	979,101.39	77.1	9,412,925.25	3,747,669.05	264.8	1,275,120.60	588,594.60	40.3	1,302,128.30	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1587.9	1,037,063.57	501,626.34	212.2	3,247,983.03	1,629,601.15	460.5	343,514.58	171,757.24	25.9	1,642,165.85	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	2,530,801.09	1,015,479.44	65.8	13,210,187.64	4,893,752.12	373.4	1,237,508.10	468,652.35	39.2	1,882,362.10	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	1,474,245.36	713,674.21	72.6	8,680,178.53	4,149,670.95	360.2	2,371,131.26	965,663.18	139.7	818,942.63	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	1,209,085.39	707,463.39	95.4	2,705,614.72	1,587,737.37	140.9	239,530.50	110,954.29	9.3	280,910.16	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	1,193,968.79	536,982.44	27.7	23,214,354.17	6,291,402.50	385.6	2,843,557.39	887,575.00	58.8	2,770,961.27	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	486,927.24	210,838.14	11.2	1,569,460.07	470,790.83	23.4				472,184.07	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	1,424,195.60	648,194.80	209.0	5,589,680.42	2,295,989.37	387.6	593,331.01	116,728.18	65.9	502,588.43	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	1,751,410.13	938,913.15	262.9	7,089,603.57	3,470,256.37	955.9	183,622.64	8,599.56	34.6	56,274.65	South Dakota
Tennessee	6,805,683.35	3,313,936.07	259.6	3,695,741.78	1,822,625.45	127.2	12,838,518.25	5,805,258.99	436.9	913,841.02	456,920.49	56.9	625,896.00	Tennessee
Texas	42,341,998.56	16,190,624.91	3122.8	4,466,439.28	1,746,748.39	291.3	24,603,135.06	9,993,410.74	1512.7	4,775,652.39	2,024,008.84	302.6	1,769,420.12	Texas
Utah	3,304,423.75	1,895,805.92	219.0	719,066.79	390,456.24	91.2	4,502,514.72	8,983,163.11	266.7	493,275.89	349,367.29	32.9	497,680.44	Utah
Vermont	1,922,114.16	942,769.12	74.4	145,101.06	72,550.52	6.2	2,108,819.21	998,976.84	51.6	53,364.64	26,682.31	1.4	493,000.21	Vermont
Virginia	10,035,301.48	4,801,782.43	552.5	924,582.46	421,537.81	29.2	9,998,047.20	4,653,970.57	352.6	747,489.36	353,972.26	28.3	361,689.93	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	444,972.29	198,980.09	29.4	3,963,074.35	1,823,200.00	147.8	76,148.13	38,000.00	5.2	535,602.46	Washington
West Virginia	5,489,747.95	2,365,041.53	256.6	1,232,116.91	562,933.47	42.8	4,619,497.76	2,069,387.81	156.0	77,099.55	27,448.00	0.3	729,321.19	West Virginia
Wisconsin	18,763,903.15	7,441,033.57	1325.3	1,018,641.46	481,571.65	49.9	4,326,468.60	2,083,976.04	178.1	135,408.28	67,703.00	7.2	3,604,166.74	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	992,457.57	581,593.73	119.1	4,412,501.34	2,784,694.56	293.6				242,964.01	Wyoming
HAWAII													365,625.00	HAWAII
TOTALS	$ 549,655,391.27	$ 237,852,399.82	32452.9	$ 60,987,768.96	$ 28,422,560.05	3572.3	$ 412,262,564.14	$ 185,313,014.62	18745.1	$ 38,130,283.37	$ 15,155,288.75	1949.3	$ 58,381,736.76	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 101,084,663.27 Federal aid $ 45,733,548.42 Miles 4,355.0

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publciation to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

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Vol. 11, No. 10, D–15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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VISION—
in SUPERVISION

A SURETY COMPANY is authority for statistics recently published which seem to confirm the popular belief that highway construction is a hazardous business venture. It is shown that the losses of bond houses on high-type road construction contracts constitute a large percentage of the premiums paid. We assume the figures are correct, although we must confess they depict a far more serious situation than we had believed existed.

BUT WHAT INTERESTS US more than any post mortem figures are the reports received at regular intervals by this bureau from its district engineers, which attribute to bad management one out of every four cases of unsatisfactory progress on Federal-aid projects. Only one other cause appears more frequently—the weather.

NEITHER CONTRACTORS NOR ENGINEERS can be expected to accept responsibility for the weather, although it does appear that a greater amount of time is lost on account of unfavorable weather conditions than there is warrant for in the conditions themselves.

BUT IT DOES SEEM that the cases ascribed to bad management are distinctly up to the contractors; and the evidence of the Federal-aid reports is reinforced by the studies of grading operations recently made by the bureau and published in PUBLIC ROADS. When it is shown that 100 of the 180 seconds required to load, turn, and dump wheelers on an average job represents unprevented preventable delay one is inclined to the opinion that highway construction would be a much less hazardous venture if contractors would put more vision into supervision.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 11 ▽ JANUARY, 1925

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A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 5, NO. 11 JANUARY, 1925 H. S. FAIRBANK, Editor

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THE SUPPORTING VALUE OF SOIL AS INFLUENCED BY THE BEARING AREA

By A. T. GOLDBECK and M. J. BUSSARD, United States Bureau of Public Roads

WHEN a given unit of load is applied to a soil over various areas, the depth of penetration is directly proportional to the square root of the area over which the load is applied. This relation has been established independently and practically simultaneously by experiments conducted in June, 1923, by the Bureau of Public Roads at the Arlington Experimental Farm, Arlington, Va., and by A. Bijls, chief of highways and bridges, of Belgium. The test conditions under which this relation has been established have necessarily been restricted and the applicability of the relation to more general conditions remains for further investigation.

By the use of formulae or diagrams expressing the established relation it is possible within limits to predict or determine from a knowledge of the penetration of any given unit load, applied experimentally over a given area on any soil, the penetration which will result from the application of the same unit load to any area of the soil, provided the penetrations involved be small enough to permit the supporting mass to act for the most part elastically.

The application of such a relation to the design of roads and more particularly to the design of footings of bridges and other structures is obvious. Its use should make possible a more certain design of footings and should be especially helpful in the prevention of unequal settlement of all kinds of structures.

The facts discovered indicate that such unequal settlement is likely to occur as the result of methods of footing design, heretofore followed universally, in which the areas of the footings under various parts of a structure are predicated upon the delivery of a uniform unit load to the soil regardless of the areas of the footings. The indications are strong that this practice is erroneous; and it has apparently been positively proved that, in its place, a method should be adopted which, based upon this observed relation of penetration and area, assures equal penetration instead of equal unit load.

The general form of the relation expressed mathematically is:

$$\frac{p}{P}=\sqrt{\frac{a}{A}}$$

in which, p and P are the penetrations resulting from the application of a constant unit load to areas a and A.

The above relation was established experimentally and was also developed mathematically on the assumption that for any particular soil, the areas under pressure at different depths are confined within lines of like inclination extending from the boundaries of the bearing areas.

DESCRIPTION OF EXPERIMENTAL METHODS

Since soils exist commonly in the nature of semi-fluids, deforming in part elastically yet yielding under the continuous application of load, it is necessary, in order to establish a basis of comparison between the supporting value of different areas, to study their behavior in one of two ways; (1) by observing the relative loads necessary to produce common penetration; or (2) by observing the relative penetrations caused by the same unit of load.

The latter method was used in the experiments of the Bureau of Public Roads; and a criterion for the measurement of supporting value was arbitrarily assumed as the unit of load which would cause a bearing block 9 square feet in area to penetrate the soil 0.1 inch.

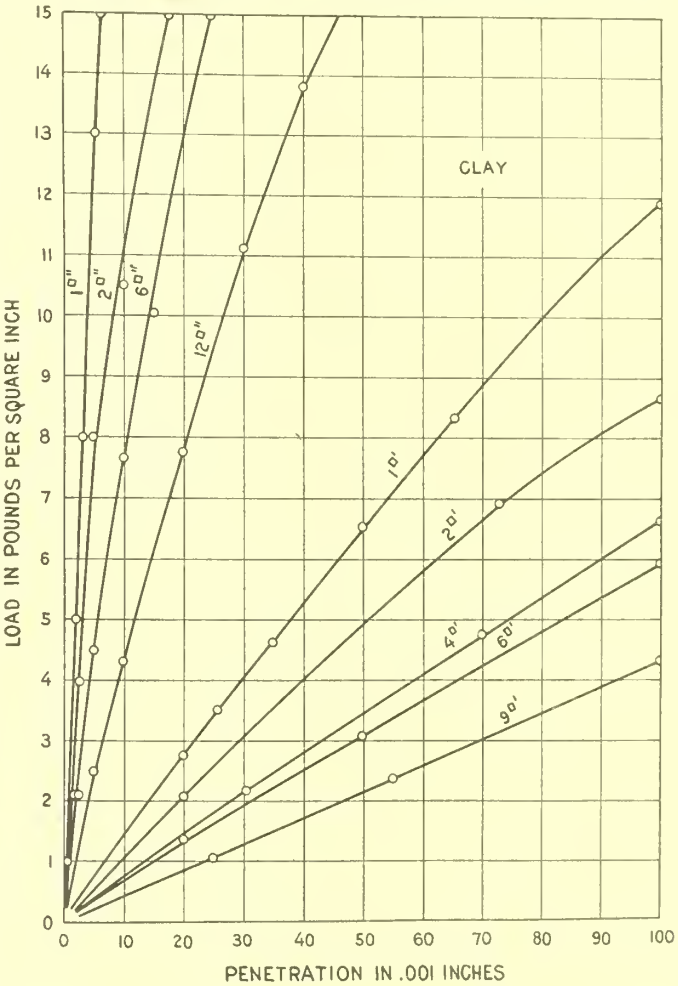

FIG. 1.—Bearing value curves for clay soil

The materials used for the experiment consisted of quantities of clay soil to which varying proportions of Potomac River sand were added. Five distinct mixes were used in which the proportions of Potomac River sand and clay soil were, respectively, 5 to 1, 5 to 3, 1 to 1, 1 to 3 and 0 to 1. The material was moistened and mixed by hand, then placed in a test bin 6 feet square, and tamped in 6-inch layers to a total thickness of 30 inches. A series of circular bearing blocks ranging in area from 1 square foot to 9

26585—25†——1

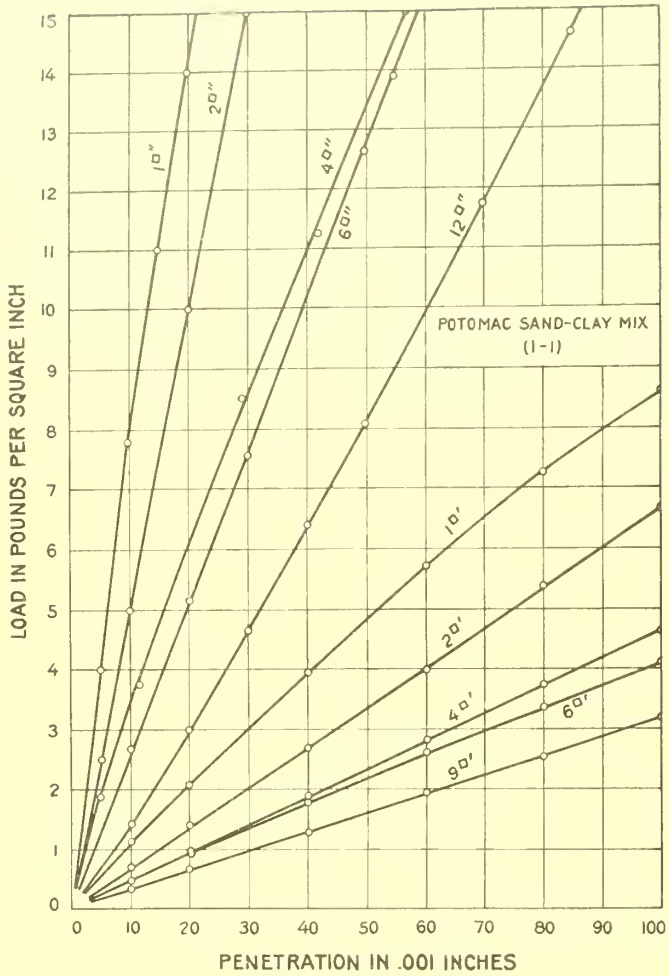

FIG. 2.—Bearing value curves, clay soil mixed in equal parts with Potomac River sand

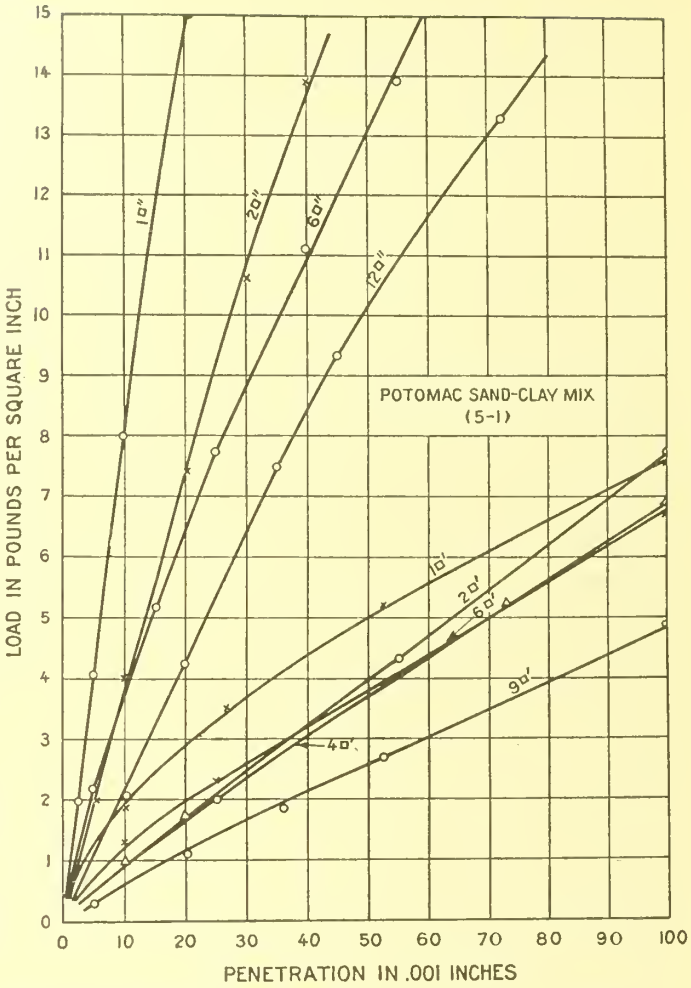

FIG. 3.—Bearing value curves for Potomac River sand and clay soil mixed in proportion of 5 to 1

square feet, were each loaded at a uniform rate by means of a hydraulic jack and simultaneous readings of load and penetration were made by observing the movement of two Ames dials placed at each extremity of a diameter. The average between the two readings was computed in each case, to eliminate possible errors due to the uneven penetration of the block. Load readings were obtained by observing the deflection of a calibrated resistance beam.

A series of smaller areas ranging from 1 square inch to 12 square inches in the form of circular metal bearing blocks was also tested. Both series of blocks were tested with each mix in the same manner. The test bin was emptied after each individual test, and the material was loosened, replaced and retamped. This was repeated a number of times with each area and with the different mixes, and the moisture content was determined each day from representative samples taken at various depths, immediately after the tests.

Figures 1, 2 and 3, represent what might be termed the bearing value curve or stress-strain diagram for all the areas on three of the mixes. It is well to note the relative positions of the curves, which undoubtedly indicate that with few exceptions the smaller the superimposed area, the greater is the unit of load required for equal peneteration.

Figure 4 shows the total loads applied to various areas, which produce a common penetration of 0.1 inch. The curves in the upper portion of the figure show that the unit of supported load increases as the superimposed area decreases for a given penetration.

The algebraic equations of the curves were derived in every instance, the reason being that there was necessity for a means of identification and comparison between the behavior of the various areas when used with different mixes. It is seen, however, that they are all of the same general type, differing in their constants, owing to the difference in compaction, moisture content, state of preparation, etc.

THE RELATION OF PENETRATION AND AREA UNDER CONSTANT UNIT LOAD

Since those in charge of the tests were at the time concerned with the load causing a penetration of 0.1 inch for an area of 9 square feet, and contemplated that this investigation would make it possible to obtain this information by means of tests performed with a bearing block of small area, it was necessary to analyze the data from a different point of view, namely, by considering the relative penetrations of the areas produced by a constant unit of load, the unit of load in each case being that which caused 0.1 inch penetration when applied to an area of 9 square feet. Therefore curves similar to Figure 5 were plotted from data obtained on each mix. Since they were found to be of similar type, the existence of an unknown general relation was suspected. Consequently, the curve shown in Figure 6 was developed from the average of the five individual curves. It is evident from this average curve that

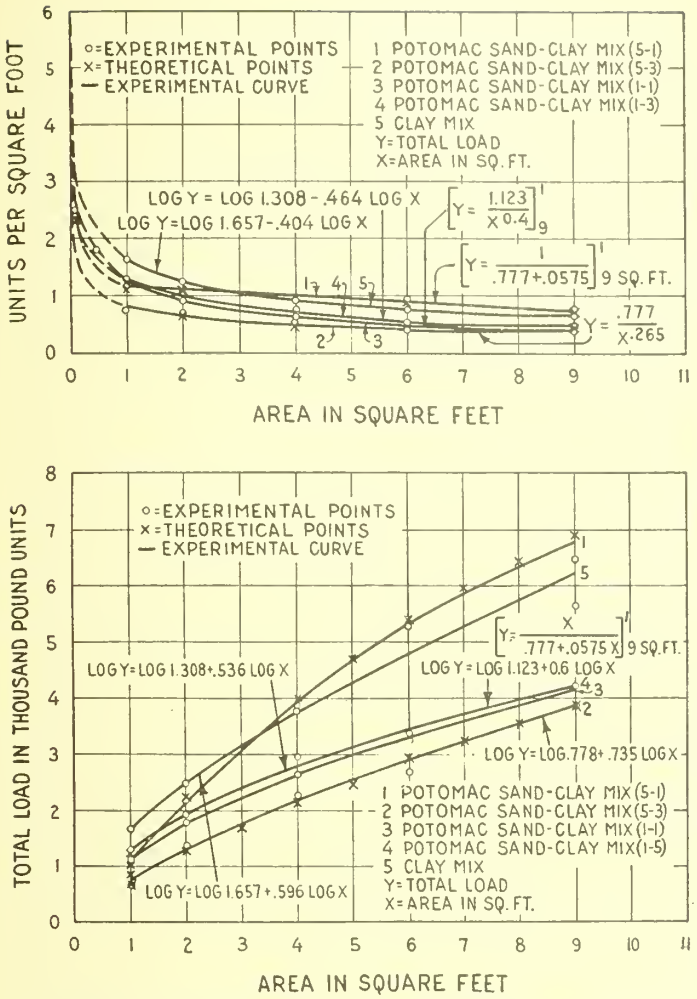
o=EXPERIMENTAL POINTS
x=THEORETICAL POINTS
— EXPERIMENTAL CURVE
1 POTOMAC SAND-CLAY MIX (5-1)
2 POTOMAC SAND-CLAY MIX (5-3)
3 POTOMAC SAND-CLAY MIX (1-1)
4 POTOMAC SAND-CLAY MIX (1-3)
5 CLAY MIX
Y=TOTAL LOAD
X=AREA IN SQ. FT.
LOG Y = LOG 1.308 - .464 LOG X
LOG Y=LOG 1.657-.404 LOG X
Y= 1.123 / X^0.4
Y= 1 / .777+.0575
9 SQ. FT.
Y= .777 / X.265
UNITS PER SQUARE FOOT
AREA IN SQUARE FEET
o=EXPERIMENTAL POINTS
x=THEORETICAL POINTS
— EXPERIMENTAL CURVE
Y= X / .777+.0575 X
9 SQ. FT.
LOG Y=LOG 1.308+.536 LOG X
LOG Y=LOG 1.123+0.6 LOG X
LOG Y=LOG.778+.735 LOG X
LOG Y=LOG 1.657+.596 LOG X
1 POTOMAC SAND-CLAY MIX (5-1)
2 POTOMAC SAND-CLAY MIX (5-3)
3 POTOMAC SAND-CLAY MIX (1-1)
4 POTOMAC SAND-CLAY MIX (1-5)
5 CLAY MIX
Y=TOTAL LOAD
X=AREA IN SQ. FT.
TOTAL LOAD IN THOUSAND POUND UNITS
AREA IN SQUARE FEET

FIG. 4.—Total and unit loads required to produce common penetration of 0.1 inch under different areas on various soils

under a constant unit of load the penetration decreases very rapidly as the area decreases, apparently according to some determinate and continuous mathematical function.

The curve of experimental results shown in Figure 6 by the dotted line is very closely approximated by a parabola shown by the solid line whose general equation is

$$y = ax^2$$

where y = area of bearing block in square feet
x = penetration in inches
a = a constant which depends upon the penetration of a given size of bearing block. In the present analysis the relation is based on the penetration of a 9 square foot block to a depth of 0.1 inch. For this condition a becomes 900.

The relation between area of bearing and penetration, for a constant unit load, corresponding to that which causes a 9 square foot area to penetrate 0.1 inch is thus expressed by the following equation:

$$y = 900\ x^2$$

For the conditions of this test it is apparent that the penetrations of several areas are proportional to the square roots of the areas. Necessarily, the tests have been limited to a range of bearing areas which does not include the larger areas encountered in structures, the

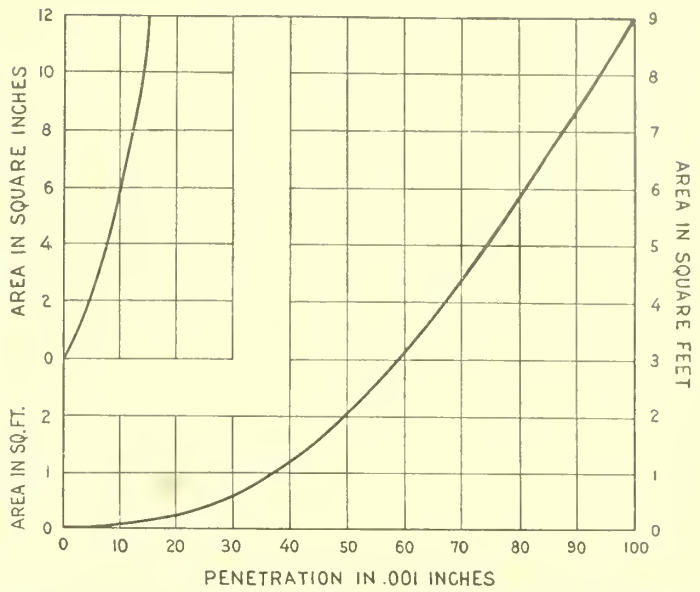

FIG. 5.—Typical curve showing penetrations of various areas when loaded with the unit load which caused penetration of 0.1 inch when applied to area of 9 square feet

largest area used being 9 square feet; moreover, the soil was confined within a bin. The above relation having been established for the range of areas and test conditions as noted, it will be interesting to investigate the possibility of this same relation holding true for other areas and conditions.

Curves showing the distribution of earth pressures through a sand fill due to load application on bearing blocks placed on the top of the fill will shed light on the possible reason for the above relation and will also furnish the basis for assumptions which will be made in the development of a general theory.

The curves in Figure 7, drawn from experimental results obtained by the Bureau of Public Roads in 1917,[1] illustrate the variation in pressure distribution at various depths below the area of load application.

[1] See 1917 Proceedings of the American Society for Testing Materials.

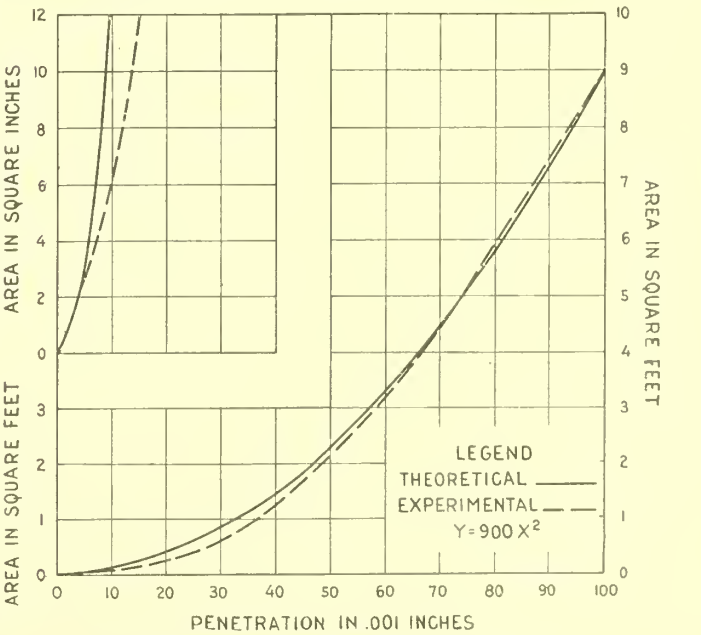

FIG. 6.—Comparison of experimental and theoretical curves

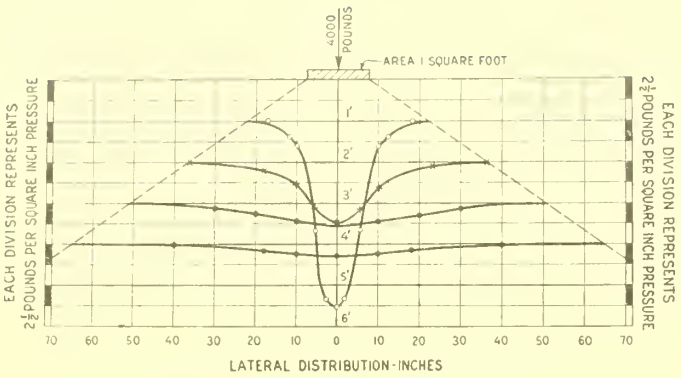

FIG. 7.—Lateral distribution and intensity of pressure in soil at various depths resulting from the application of load to the surface

In Figure 8 it is shown that at any particular depth the intensity of pressure is dependent on the area of the bearing block used, and, in fact, it seems to be the case that the maximum intensity of pressure at any given depth is in direct proportion to the area of the bearing block. It is therefore quite evident that the curves of distribution of pressure are similar for different areas, though of different magnitude and different distribution, depending on the area of load application. It is also quite evident that for like intensities of pressure applied to different areas, like intensities of pressure within the soil due to the loads on those areas will be attained at different depths in the soil, a large block compressing a greater depth of soil through a given range of pressure intensities than a small block. From this consideration it follows that for corresponding pressure intensities on blocks of different areas a large bearing block must depress the soil more than a small block. In Figure 7 it is seen that the layers under compression at different depths below the bearing block are confined approximately between lines radiating downward from the boundary of the bearing area.

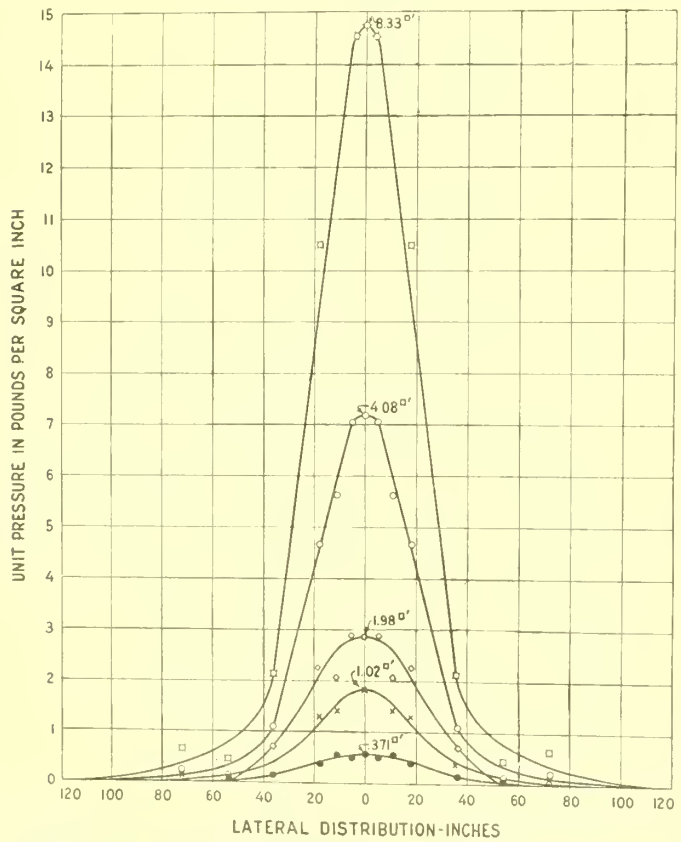

FIG. 8.—Lateral distribution and intensity of pressure resulting at the same depth from application of a constant unit load of 4,000 pounds per square foot to bearing areas of several sizes

In Figure 9 we have two areas, A and a, each subject to unit load P. Assume that the actual curves of pressure in the soil beneath these areas are replaced by equivalent uniform pressures confined within areas as indicated. Let P_1 be the intensity of pressure produced on area a_y at depth y by area a. Also let y_1 be the depth to area A_{y_1} where the intensity of pressure produced by A is equal to P_1. Then, since reactions at any depth must equal applied loads,

$$Pa = P_1 a_y \text{ and } PA = P_1 A_{y_1} \text{ or}$$

$$\frac{Pa}{PA} = \frac{P_1 a_y}{P_1 A_{y_1}} \text{ or}$$

$$\frac{a}{A} = \frac{a_y}{A_{y_1}}$$

By similar triangles—

$$\frac{r}{R} = \frac{y_r}{y_R} = \frac{y}{y_1} \text{ or } y_1 = y\frac{R}{r}$$

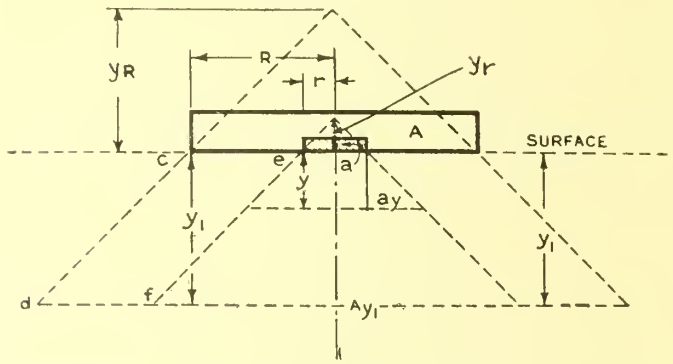

FIG. 9.—Diagram showing distribution of pressure from two bearing areas and depths to planes of equal pressure intensity

Assuming a constant modulus of elasticity, E, the total deformation, e, of a column of soil of length l at unit stress S is expressed as follows:

$$e = \frac{Sl}{E}$$

Applying this relation to the problem in hand, since the total deformation must be the penetration of the bearing blocks in any case, the ratio of total deformations or penetrations under bearing blocks of the two areas assumed must be:

$$\frac{e_a}{e_A} = \frac{\frac{Sy}{E}}{\frac{Sy_1}{E}} = \frac{y}{y_1} = \frac{y}{y\frac{R}{r}} = \frac{r}{R} = \frac{\sqrt{a}}{\sqrt{A}}$$

That is, the relative penetration of the two areas is equal to the ratio of the square roots of the areas for like intensities of pressure on the two areas. Although the above analysis is not exact, it at least tends to show a strong probability that the law which seems to be followed by the bearing areas tested may be extended to areas of any size.

(Continued on page 8.)

HIGHWAY RESEARCH BOARD PROCEEDINGS

FOURTH ANNUAL MEETING HELD AT WASHINGTON IN DECEMBER

By J. L. HARRISON, Highway Engineer, United States Bureau of Public Road

THE fourth annual meeting of the Highway Research Board was held at Washington, D. C., December 4 and 5, 1924. Dr. Vernon Kellogg, permanent secretary of the National Research Council, in the address of welcome announced that the purpose of the Highway Research Board is to affect much needed correlation in highway research and to disseminate information in regard to the results of such research.

Comparing the development of the modern dye industry the phenomenal progress of which has been based upon scientific research with the slow progress that has been made in certain other industries and arts, such as ceramics, in which reliance has been placed upon rule-of-thumb methods, he sounded the keynote of the board's sessions when he stated emphatically his firm belief that research yields large dividends in actual dollars and cents and that both economy and progress depend upon it.

Thomas H. MacDonald, chief, Bureau of Public Roads, one of the directors of the Highway Research Board, discussed the financial value of highway research, pointing out that a discussion of such a topic ought to introduce a note of practical application of those scientific principles and facts, which we arrive at through research and through investigation, and bring out their actual value in dollars and cents to the people of the United States.

That it does actually have such a value he illustrated by a number of examples. One of these—the use of the thickened edge in the design of concrete roads—which is the result of an extended research, makes possible a saving of about 390 cubic yards of concrete per mile. At $10 a cubic yard this is a saving of $3,900 per mile. There are 24 States that have adopted this new design. If, in these States, 2,400 miles of concrete roads are constructed each year, the saving effected amounts to $9,360,000. Other equally important savings from improved design will follow, Mr. MacDonald stated, as research work is developed.

Another case in which it is known that decided savings have resulted from highway research is the intensive laboratory research by which the Iowa Highway Commission has been able to develop means of appropriately using nonstandard aggregate in concrete construction. In building approximately 75 miles of concrete pavement during 1920 and 1921, $155,638 were saved to the State through the successful work of its laboratory. Such savings can be made in many States if the attention of their laboratories is turned toward research work.

Another phase of the value of highway research is presented in the construction field. At present there is a common feeling that engineers are crowding many highway contractors to the wall. It is the duty of highway engineers to analyze this attitude and take steps to overcome it. But this can be done only through research—research particularly in the construction field.

Mr. MacDonald pointed out the causes which have been found to prevail where there has been unsatisfactory progress on Federal-aid road work, of which the two most important were: weather, 33 per cent; and management, 27 per cent. He presented graphs showing how contractors lose money on such operations as wheel-scraper work and how effective rather simple suggestions can be in helping contractors to do their work more effectively. In one case cited the cost of grading work was reduced more than 20 per cent through suggestions that it was possible to make to contractor as the result of research in the construction field.

"In my judgment," said Mr. MacDonald, "the whole story in profitable and unprofitable highway contracts lies in that operation, and it seems to me that in this field the research agencies have an opportunity not only of helping the contractors who are building our roads to increase their profits, and the quality of their output, but actually to decrease the cost of highways to the public. When we are spending in this country each year a billion dollars or more for highway construction and maintenance, is it necessary to develop any farther the enormous financial value of highway research when, on a single operation a saving of between 20 and 25 per cent can be made by the use of stop watches and the analysis of the causes of delays and how to avoid them?"

In rendering his report, Charles M. Upham, director of the Highway Research Board, drew attention to the fact that economy in the construction and maintenance of highways is necessary but that economy does not mean spending less money or restricting highway development. Rather it is to be attained by making more complete use of advanced ideas in methods, practices and designs as these are developed through research and investigation. Emphasis was laid on the fact that one reason for the common failure to bring highway construction and maintenance practices into more complete harmony with the results of completed researches is the fact that as presented to the profession these are commonly accompanied by such a mass of supporting data that the busy executive finds it difficult to give them the study they deserve. Data published in the form now commonly used are valuable. But, in the interest of economy, they should also be presented in condensed summary reports. Such reports would summarize the important facts of the fuller reports, but would give sufficient details to afford a proper understanding of the results. Perhaps the obstacles that now interfere with the spreading of knowledge of research work might be overcome by the publication of consolidated and condensed reports by the Highway Research Board acting in cooperation with other agencies. The large losses in the highway field which can be traced to an incomplete dissemination of information with regard to the results of important researches emphasize the value of such a service.

REPORT OF COMMITTEE ON ECONOMIC THEORY OF HIGHWAY IMPROVEMENT

In representing his report, Prof. T. R. Agg, chairman of the committee on economic theory of highway improvement, touched on the scope of the committee's work, and drew attention to the fact that research in

this field involves studies of the road surface and the cost of operating vehicles. The determination of economic relationships must include studies of the cost of constructing and maintaining each type of surfacing, as well as the cost of operating traffic over it.

The cost of operation has to be reduced for study to the factors entering into it, such as the wear and tear on the vehicles used, gasoline and oil consumption, tire wear, etc. In turn, gasoline and oil consumption are affected by the rolling resistance which must be overcome by the engine in driving the car. Test results show a variation of from 20 to 700 pounds per ton in rolling resistance at a speed of 10 miles an hour. Except on a smooth pavement, at slow speeds, there appears to be no important difference in rolling resistance between low pressure tires and standard tires. Gasoline consumption may also be affected by gradient. Experiments show that the rate of tire wear is affected by the type of surfacing. It also appears to be affected by the amount of inflation.

By such studies as those on which reports have been rendered, as here briefly referred to, and those under way or proposed, a considerable body of data on the cost of operating traffic on various types of surfacing is being built up.

COMMITTEE ON STRUCTURAL DESIGN OF HIGHWAYS

A. T. Goldbeck, of the United States Bureau of Public Roads, chairman of the committee on structural design of highways, submitted his report in several sections, the first of which dealt with the various investigations in progress in connection with the determination of the characteristics of subgrade soils and the development of methods of improving bad subgrades.

Subgrade investigations.—It is believed that the introduction of such granular materials as sand, gravel, cinders, etc., in the subgrade not only tends to reduce the capillary tension in the subgrade soil but allows condensed moisture to be deposited on a subgrade made up of materials the bearing value of which is not so greatly decreased by moisture, but as yet no very definite relation has been determined between capillary moisture, as determined in the laboratory, and the maximum moisture content which may be found in the subgrade under a pavement.

Design of concrete slab.—The conclusions from a rather elaborate series of tests on the effect of static loads on road slabs were presented. Among the more important facts brought out, the following appear:

Plain concrete of 1:3:6 mix offers resistance to impact ranging from about 60 per cent to 80 per cent of that developed by plain concrete of 1:1½:3 mix. In all cases the slabs tested appear to offer a greater resistance to impact than to static loads.

From the studies of motor truck impact, it appears that little impact is developed by pneumatic tires even on rough granite block pavements. Solid tires, on the other hand, develop stresses ranging from 200 to 500 per cent and more of the static load stresses. Cushion tires, depending on their design, develop impact stresses intermediate between those developed by pneumatic tires and those developed by solid tires.

The tests made on skew arches show that higher reaction stresses will exist at the obtuse angle end of the abutment than at the acute angle end. Some idea of the variation in stress between the two ends of the abutment can be had from the fact that for a 60 degree skew arch both the horizontal and the vertical reactions at the acute angle end are practically zero.

Fatigue in concrete.—An extended paper on the fatigue of concrete was presented by Dr. W. K. Hatt of Purdue University. The divergency of opinion which has existed as to the elastic limit of concrete was noted and discussed. The conclusion drawn was that this divergence is probably due to the use of imperfect instruments, to the age of the samples tested and to the amount of moisture present in the samples. For tests in this field the use of concrete which has aged from nine months to a year was recommended.

The investigations of fatigue which were reported indicate that for specimens over six months old the endurance limit under repeated stress is about 55 per cent of the static load capacity. Specimens aged 28 days can not be depended upon to have an endurance limit of over 40 per cent. Rest periods do not appear materially to affect the fatigue limit of well-cured concrete. Stresses above the fatigue limit cause progressive deterioration. From this it would appear to be desirable to so design concrete highways that the maximum loads which they must carry will cause maximum fiber stresses not to exceed 50 per cent of the modulus of rupture.

COMMITTEE ON HIGHWAY FINANCE

H. R. Trumbower, of the Bureau of Public Roads, chairman of the committee on finance, stated that, in considering highway finance, one of the first problems which arises is the selection of an appropriate basis for the apportionment of the cost of construction and maintenance among those benefited directly and those benefited indirectly. This leads at once to a study of motor vehicle traffic and the taxes now imposed on this traffic. Of these, the license fees and the gasoline taxes go almost entirely into the highway funds and, for 1924, are expected to provide one-third of the money expended on construction and maintenance. This proportion is considerably greater than it was in 1921 when only about one-seventh of the money expended on highway construction and maintenance was collected in this way and this increase indicates the prevailing tendency toward the levying of heavier taxes on motor vehicles. To determine whether this tendency is proper and should be encouraged will require investigation in such fields as the effect of improved roads on adjacent land values, relative use of city streets and rural highways, personal property taxes on motor vehicles, etc. Only after careful studies in these and in related fields will it be possible to determine whether the present scheme of taxing motor vehicles makes for a thoroughly equitable and satisfactory distribution of taxes among those benefited.

In his discussion of highway finance Frank Page, chairman of the North Carolina State Highway Commission, presented an interesting analysis of contracts defaulted in North Carolina. On a $75,000,000 construction program involving 700 contracts, 35 failures involving 21 contractors have been recorded. Nineteen of these 34 contracts were in the hands of contractors devoid of experience. They had practically no equipment which, of course, means that they had no funds. Five of the contractors had experience but had failed in other States and so had come to North Carolina without funds. Six of the failures were due to neglect of business. Two of the contracts were in the hands of "ordinary, everyday crooks" who "went onto the job with the full determination to milk it." The last two failed because they underbid.

Mr. Page expressed the view that more care should be exercised by the companies bonding contractors. The present practice of making the premium large enough to enable bonding companies to carry losses of this sort is unfair to the good contractors and unfair to the State.

A lively discussion followed the presentation of this paper indicating that the present practices in regard to selecting risks, calculating premium rates, and taking care of defaults involve many problems which deserve careful and thoughtful investigation.

REPORT OF COMMITTEE ON HIGHWAY TRAFFIC ANALYSIS

G. E. Hamlin, of the Connecticut Highway Commission, chairman of the committee on highway traffic analysis, stated that the automobile saturation point is a matter of great importance in the determination of future traffic, but added that it appears that this point can not be determined on the basis of any existing data.

The importance of a study of the question of adequate rights of way to protect future traffic developments was urged. "Bottle-neck" points on existing highways often cause a reduction in traffic-carrying capacity. Highway engineers should study and eliminate these. Minimum speed laws are needed to develop the full capacity of highways. The probability that the larger part of the accidents occurring outside of urban districts are directly chargeable to slow-moving vehicles is another reason for suggesting minimum speed laws for heavy-traffic roads. More attention to the relation of the highway wearing surface to traffic needs was stressed. Many miles of wearing course capable of carrying traffic 365 days in the year are needed rather than a few miles of high-class pavement.

The purpose of highway transport surveys was discussed. These should (1) determine daily, seasonal, and yearly traffic and its distribution over the State, (2) estimate future traffic, and (3) show the relation of traffic to traffic factors. From the data so secured highways should be classified and appropriate widths for the various classes developed.

The discussion was led by John N. Mackall, chairman and chief engineer of the Maryland State Roads Commission. Mr. Mackall invited attention to the following quotation from Wellington, "* * * the distorted preeminence given by engineers * * * to the pettiest details of how to build the separate works which go to make up a highway, to the neglect of the larger questions of where to build and when to build and whether to build at all, has in it something that is at once astonishing and discouraging * * *." The real test for a highway transport study, said Mr. Mackall, is: "Does such a survey furnish road authorities with data that assist in the proper location and design of new highways?" To meet this need fully, traffic studies should develop rating curves from which to determine the relative meaning of coincident and subsequent traffic counts. Such curves would form an adequate basis for determining maintenance cost fluctuations and would serve as the first step in solving the larger problem of making dependable estimates of traffic to be expected on proposed improvements.

The second step in answering the larger questions of when to build, where to build and whether to build at all, questions which must be answered if real economy is secured in the development of any large highway program, involves the determination of how the traffic is likely to be affected by those factors known to affect traffic. Most of these factors, such as the proximity of large cities, manufacturing enterprises, points of historical interest, touring, etc., are well recognized but no rules for determining their effect on the traffic on proposed highways have, as yet, been developed.

Finally, rational rules for assigning value to the service rendered to highway traffic are needed. Valuable contributions have been made in this field, but it can not fairly be said that there now exists any uniformly recognized standard by which to determine whether the value of a proposed highway improvement will exceed the value of the unimproved road that it may replace by enough to justify the cost of its creation.

The objective is the development of a basis for the rational statement of economic values which will enable responsible engineers to obtain estimates of service value that are of the same order of merit as the estimates of cost now so commonly used.

Prof. A. H. Blanchard of the University of Michigan spoke on the topic, "Comprehensive research program justified because of increase in efficiency in highway improvement and economic utilization of highway transport." Professor Blanchard showed that various economies in design and in the selection of materials result from research but that one of the greatest savings accrues to highway transport which is benefited both by reduced cost of operation and by increased comfort and convenience.

REPORT OF COMMITTEE ON CHARACTER AND USE OF ROAD MATERIALS

H. S. Mattimore, of the Pennsylvania State Highway Department, chairman of the committee on character and use of road materials, reported that the committee's investigations indicate that the durability of concrete is affected by the durability of its constituent aggregates. It also appears that the nature of the aggregates may be the principal factor in expansion failures in concrete road slabs and bases. Other items of importance mentioned in the report are that the Jones-Talbot rattler test does not appear to be a reliable test for wear; that proportioning of concrete aggregates by weight seems to have merit; and that accelerated tests of aggregate for soundness are advisable.

As a part of the discussion of this report, W. E. Hawkins, maintenance engineer for the North Carolina State Highway Commission, presented a paper on "The economic selection of road building materials." Mr. Hawkins drew attention to the fact that most States possess large supplies of materials which do not meet the generally accepted standards, and so can not be appropriately used under standard specifications. Under such conditions it is the duty of engineers to make careful studies not only of the available materials themselves but of the results of unusual proportions and methods to ascertain whether practices can be devised which will justify their use. Such a program involves the use of laboratory facilities constructively, that is, for the determination of whether to use and how properly to use available materials, which represents an advance over the older practice of using laboratory facilities for the determination of whether materials of construction comply with pre-established standards.

REPORT OF THE COMMITTEE ON MAINTENANCE

The report of the committee on maintenance, W. H. Root, of the Iowa Highway Commission, chairman, called attention to the importance of snow removal, and added that more attention is required on the side of economy. Figures collected indicate that plows should be pushed rather than pulled; that ordinary push plows do not perform satisfactorily where the snow is over 24 inches deep and that improved V-type plows and rotary plows are to be had and should be used where heavier snowfalls are to be expected. A great deal of work can be saved by the erection of snow fences.

In the field of maintenance accounting it was recommended that highway officials adopt a standardized distribution of expenditures. Three headings were suggested, as follows: (1) Maintenance administration; (2) maintenance proper; and (3) betterments.

In his discussion of the committee's report, under the topic "The necessity of research to reduce the cost of highway maintenance," W. H. Connell, deputy secretary of highways and engineering executive, Pennsylvania State Highway Department, emphasized the need of more effective organization and of the application of standards of output and of cost per unit of output in developing real efficiency in maintenance. He also laid emphasis on the need of more complete recognition of the fact that restorations and replacements are strictly maintenance operations.

C. A. Hogentogler, of the Highway Research Board, read an interesting statement on the plans for a fact-finding survey on the economic value of reinforcing in concrete roads, which is being conducted by the board. In this study an effort is being made to determine from a survey of existing roads, the influence of steel reinforcement as affected by traffic, subgrade and climatic conditions, the conditions under which reinforcement is especially beneficial, the effect of slab design on the efficiency of reinforcement, and the relative cost of plain and reinforced concrete roads considering the initial investment, annual maintenance, and renewal charges.

The executive committee selected for the ensuing year is as follows: A. N. Johnson, chairman; W. H. Connell, vice chairman; Charles M. Upham, director; T. R. Agg, A. J. Brosseau, H. C. Dickinson, Thomas H. MacDonald, William Spraragen.

A complete report of the proceedings of the fourth annual meeting of the Highway Research Board may be secured by addressing Charles M. Upham, director, Highway Research Board, National Research Council, 2101 B Street NW., Washington, D. C.

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THE USEFULNESS OF THE RELATION

By application of this established relation, it is possible to make observations on the penetrations and corresponding loads of a small test area and to interpret results therefrom in terms of the supporting power that can be expected from a larger area. If a supporting mass of subgrade can be accurately reproduced in the laboratory and an accelerated supporting value test can be performed upon it with a small area, then its supporting value in the field under proposed or existent superimposed areas can be predicted or determined. This relation should also be valuable in predicting the penetrations of large areas when the penetration of a test area is known.

In connection with the results of this investigation, attention is called to the conclusion drawn by A. Bijls, chief of highways and bridges, of Belgium. An extract from his article on results obtained from an investigation which he conducted follows:

THE INFLUENCE OF THE SIZE OF SURCHARGED AREA [2]

The great influence of the size of surcharged area should be considered as one of the most important results of these experiments. When the area increases the elastic compression increases but E_o decreases.

One might ask why this has not been established before. It is necessary to find the cause in the fact that, undoubtedly there never has been an attempt made to separate systematically the elastic compression from the permanent compression.

There probably exists a sufficiently definable relation between supporting power and permanent deformation; this force is probably independent of the elastic deformation.

For the permanent deformations, we may refer to the permanent slipping of soil particles. This necessitates a shift of equilibrium within the supporting mass of the soil; also, the increase of permanent deformation will constitute, in general, a certain indication that we are approaching the limit of the supporting power.

Since the mass, under a surcharged area, is seldom found largely in a definite state of equilibrium, it is in general impossible to establish the condition under which it exists in the field. We can not therefore make absolute theoretical deductions and one must have recourse to more fundamental hypotheses.

We can therefore assume that the permanent compressions are produced by displacements of sand grains and that elastic compressions are the result of elastic deformations or slipping, and in turn, that the pressure is transmitted under an angle of 45°.

Upon these hypotheses, the author arrives at the conclusion that the elastic compression of a surcharged area is proportional to the side of the area or to the square root of A, if the area is not square. The experiments seem to confirm this result.

[2] Extract from article in "Le Genie Civil" of May 26, 1923, by A. Bijls, chief, highways and bridges of Belgium.

RECENT CONCLUSIONS IN HIGHWAY RESEARCH

By A. T. GOLDBECK, Chief, Division of Tests, U. S. Bureau of Public Roads

TO one in a position to survey the whole of the wide field of highway research and appreciate the relations of the various efforts made by numerous agencies one to another, there has come within the last year a very pronounced sense of a gradual convergence of the facts brought out by these efforts toward a well defined and rational process of highway design. Viewing the various single experiments separately one is apt to lose the sense of their position in the entire scheme and to miss the full appreciation of their value. It is therefore a good thing to stop for a moment occasionally and review the whole field, and obtain a new orientation to guide our further progress.

In attempting such a review at this time it may be well to consider not only the definite conclusions that have been reached, but also to include certain well defined indications which may later resolve themselves into conclusions. Through a knowledge of these indications it may be that much valuable time will be saved in the process of making the results of research a part of everyday practice.

THE SUBGRADE

All highway engineers have observed the fact that the soils encountered in highway construction are variable in their behavior. They have observed that some of these soils shrink and crack when they become dry and swell upon taking up moisture; that some of them absorb large quantities of moisture by capillarity from below and are utterly incapable of carrying load when in a moist condition, while others, retaining uniformly low moisture content, are very stable, change only slightly in volume, and are affected little in their load-carrying ability. The characteristics of the various soils shade from one of these extremes to the other, and the question naturally arises as to what is the dividing line between good and bad soil characteristics from the highway engineer's standpoint and how should he take account of these variable characteristics of soils in highway design.

One of the first steps toward an answer to this question was to develop suitable tests in the laboratory for classifying soils according to characteristics which would be significant to the highway engineer; and we may now say definitely that we have succeeded in developing suitable tests for differentiating the various shades of quality in soils. The details of the tests developed have been described elsewhere. They include determinations of shrinkage, moisture equivalent, water absorbed by capillarity, slaking value, dye adsorption and bearing value, as well as the more familiar mechanical analysis.[1] A combination of some of these will serve to identify the essential characteristics of a subgrade material.

We have reached the stage where, for all practical purposes, we can conduct these tests in such a way that we can get identical results with identical samples. This is an important conclusion to the testing engineer but the highway engineer wants to know what test values are the significant ones, what values indicate bad soils and what values good soils. Research has advanced a long step in this direction but a true evaluation of the test results of subgrade samples must ultimately be arrived at through the coordination of a large number of laboratory tests with field behavior. Such coordination is a difficult task, for in addition to the influence of the subgrade in causing success or failure of a road surface, the coordinator must recognize and eliminate the influence of a large number of factors such as weight of traffic, condition of drainage, strength of materials, method of construction, and suitability of the design of the road surface for the traffic carried. By and large, however, it has been possible to make a step in the direction of coordinating laboratory test results with field behavior. Much credit is due to A. C. Rose, of the Bureau of Public Roads, District No. 1, Portland, Oreg., for his successful efforts toward this end. As a result of his work, the indications are very strong that when a soil has a so-called "moisture equivalent" in excess of 20 per cent or a lineal shrinkage value in excess of 5 per cent, it is apt to be unstable in that it will take up excessive amounts of moisture and consequently heave readily due to frost action or change excessively in volume due to moisture changes.

Under such conditions special precautions are necessary to insure against failure of the pavement due to subgrade movement. Thus, drainage, both surface and tile, might have to be provided to remove water in excess of capillary moisture. The pavement might, however, be strengthened by increasing the depth, by the use of steel or by the use of a layer of stable subgrade material. Sand or a similar cushion might be provided under the road surface or the soil might be treated to render it more stable by the use of admixtures.

Suitable field tests have been developed for determining the so-called "moisture equivalent" of the soil and this test combined with the shrinkage test, which might also be made in the field without elaborate equipment, gives an excellent idea as to whether or not subgrade trouble is to be expected. The ability to perform these two tests and to interpret them should be a part of every field engineer's equipment, and they should be used and the results noted, for they are highly significant.

THE IMPROVEMENT OF BAD SUBGRADES

When a bad soil is encountered in road construction the question naturally arises as to what is the best thing to do with that soil or with the design of the road under construction. Years ago railroads both in this country and abroad discovered that in order to prevent the subsoil from working up through stone ballast it was necessary to provide a layer of finely granular material. In general, cinders have been used for this purpose but so also have bank-run gravel, ashes, and in one case it has been reported that cement was mixed with the soil. A bad soil primarily is one which has the ability to take up a large amount of water by capillarity, and which swells and shrinks excessively with changing moisture content, and which also has low bearing value in its moist condition. Such properties make it a poor supporting medium for highways and when the subgrade is to be improved it should be treated in such a way that: (1) It will have low shrinkage and expansion; (2) it will have high bearing value

[1] See PUBLIC ROADS, vol. 4, No. 3, p. 15.

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at its highest moisture condition; (3) it will retain as little moisture as possible.

The sand-clay road furnishes an excellent example of the treatment of bad subgrades. Sticky, plastic clay is made into a stable nonyielding surface by the addition and admixture of sand. Cinders, or other fine granular material will accomplish the same end. These processes are so well developed that the following can be set down as an absolute conclusion:

> A poor subgrade material can be transformed into a good subgrade by the addition of sufficient fine granular material or by the use of a layer of suitable granular material.

EFFECT OF SIZE OF BEARING AREA ON BEARING VALUE

If the bearing value of the subgrade is important, the question immediately arises as to what effect the size of the bearing area has on the supporting value of the soil. The importance of the series of investigations described for the first time in this number of PUBLIC ROADS is that it apparently provides the answer to this question. As the tests themselves are described on another page it will not be necessary to describe them in detail here, or to go into these conclusions except to repeat their most important conclusion, as follows:

> When like intensities of load are applied to bearing blocks of different size the relative penetrations of these areas into the soil will vary as the square roots of the areas.

This means that a large area will penetrate more than a small area when both are loaded with the same load per square foot. This will be an important conclusion to those interested in the design of footings for structures. It applies so long as the loads and the penetrations are relatively small thus enabling the soil to act largely elastically. Its application to large areas resting on a subgrade of unlimited depth has yet to be demonstrated.

BLANKET COURSES

In many States it is still the practice to build macadam on all kinds of subgrades in exactly the same manner as when this type of construction was introduced by John Louden McAdam. A layer of rather uniform coarse stone is laid directly on the subgrade in this type of construction. Under the action of the present-day heavy loads, and under the action of the roller, the subgrade materials, especially plastic clay materials, find their way up through the voids in the stone. The stone is thus lubricated with moist clay and the surfacing is thereby rendered much less capable of supporting loads because its effective depth is greatly decreased.

> It has been proven not particularly by theoretical tests but rather by actual construction in a number of different localities that a blanket course of fine granular material such as cinders, bank sand or gravel is very effective in preventing the clay subgrade from penetrating upward into macadam and its resistance to loads in this way is very greatly increased.

There is nothing new about this method. It has been practiced for years but its advantages seem not to have been universally recognized and it is well to emphasize them again at this time.

DRAINAGE

High moisture content in the subgrade is detrimental to road surfaces: (1) Because the bearing value of the subgrade is greatly decreased; (2) because the effect of freezing in causing unequal heaving of the road surface is more severe; and because (3), when the moisture dries from the subgrade, this drying is accompanied by considerable shrinkage of certain types of subgrades with a resulting nonuniform support for the road surface. The amount of harm which moisture can do is, of course, closely linked with the characteristics of the subgrade material. Moisture may be said to exist in the subgrade in the form of vaporized moisture, capillary moisture and free moisture. Free water is water which will drain out of the soil if a drainage system is provided, whereas the amount of capillary moisture is unaffected by any drainage system. When soils have an ability to take up large amounts of capillary water all types of drainage seem to be ineffective in preventing these soils from becoming very wet under the road surface and generally the water is supplied by capillarity at a higher rate than it can evaporate from beneath the surface. The investigations thus far made point very strongly to the following facts:

> Water exists in the subgrade: (*a*) As free water, (*b*) as capillary moisture; (*c*) as vaporized moisture.
>
> The moisture content of the subgrade is apt to increase under low temperature conditions due: (*a*) To the condensation of vaporized moisture, (*b*) to increase in capillary moisture.
>
> Drainage systems should be provided wherever free water is likely to exist but such systems are quite generally ineffective in removing any moisture in excess of free moisture.

EXPERIMENTS IN CONNECTION WITH ROAD DESIGN

A number of investigations have been made during the past year in connection with road design. They include both impact and static load tests on road slabs of different design laid at Arlington, Va., stress measurements in concrete roads, and special observations on a reinforced concrete road known as the Columbia Pike, near Arlington, Va.

The tests conducted at Arlington were made on 124 slabs of different design, some of them laid on a wet subgrade and some on a dry subgrade. They were tested in positions which are known to give the most severe stresses, namely, at the corner and along one of the edges. The tests were made first with an impact machine delivering blows, the range of which covered the range of blows known to be delivered by actual motor trucks. The conclusions thus far reached with regard to the different slabs are as follows:[2]

> The resistance of a road slab depends in part upon the supporting value of the subgrade. A subgrade of high supporting value materially increases the resistance to impact pressure.
>
> Impact resistance of rigid slabs varies neither directly as the depth of the slab nor as the square of the depth but as some power less than two.
>
> In general, plain concrete slabs show no more resistance to impact delivered at the edge than to impact delivered at a corner.
>
> Transverse cracks and longitudinal cracks near the sides of a road slab may be caused by impact delivered at the edge of the slab.

[2] Impact tests on concrete pavement slabs, by L. W. Teller, PUBLIC ROADS, April, 1924, vol. 5, No. 2, p. 1.

Plain concrete of 1:3:6 mix offers resistance to impact ranging from about 60 per cent to 80 per cent of the resistance of plain concrete of 1 : 1½ : 3 mix. The lean mix also shows more variation in strength.

Reinforcing steel in concrete slabs, if present in sufficient amount and so placed as to receive tensile stress, adds to the resistance of the slab to impact.

Reinforcing steel placed longitudinally and transversely in equal percentages is more effective in preventing corner failures than the same amount placed in one direction.

For a given percentage of steel, small deformed rods closely spaced seem to be more effective than large deformed rods widely spaced.

There is very little evidence of cushioning by bituminous tops on concrete bases at temperatures of 90° F. or less.

In these tests there was no evidence that bituminous tops on concrete bases added to the slab strength of the base, with the possible exception of the 4-inch and 6-inch bases on the dry subgrade.

In some of the tests there was evidence that while the presence of the steel did not assist greatly in preventing the formation of the first crack it did prevent the development of the crack and the further failure of the slab.

The portions of slabs left unharmed have subsequently been tested under static load, both at the corners and along the edges with the following results:[3]

The static resistance of both the corners and the edges of rigid slabs is affected by the nature of the subgrade; the more resistant the subgrade to load, the greater the resistance of the slab and vice versa.

The resistance of rigid slabs to static loads does not vary with the square of the depth but as some power greater than one and less than two. About 1.75 is the average value, the exponent being higher for slabs on the wet subgrade and lower for those on the dry subgrade.

The corners and edges of concrete slabs of the size and thickness tested offer about the the same degree of resistance to static loads.

The presence of mesh reinforcement as employed in the slabs under consideration, does not increase the load-carrying capacity of concrete slabs but does give rise to a tendency to hold together and resist complete failure after initial or elastic failure has taken place.

Bituminous topping laid on rigid slabs does not increase the resistance of the slabs to static load.

At the ordinary summer temperatures encountered, bituminous slabs of the types tested show no slab strength. The load supporting value of such slabs is greatly influenced by the subgrade support.

STRESS MEASUREMENTS IN CONCRETE PAVEMENTS

By the use of a graphic strain gauge developed by the Bureau of Public Roads a large number of measurements of fiber deformations have been made in concrete pavements of different design in order to determine the effect of traffic loads on the bending stresses produced in the slabs. The instrument has proved to be useful for determining both static and impact stresses and furnishes a means for determining whether or not concrete roads are overstressed.

The procedure in making these measurements has been to install a number of graphic strain gauges just beneath the surface of concrete pavements, placing the instrument in positions which will permit of determining longitudinal and transverse stress in important positions throughout the slab. Heavy trucks are then run over the pavement in different positions, first with the rear wheels along the outer edge, then several feet from the edge, and finally with the wheels trailing along the center joint, if any exists. The following conclusions are drawn from these tests:

In slabs of uniform thickness the highest longitudinal stresses exist along the edge and along the center joint. They occur with wheels running directly over the edge of the slab and along the center joint. The highest tension might occur either at the bottom or the top of the slab at the edge. High tension also occurs transversely at a transverse joint. The tension developed at intermediate positions is very much smaller. It is therefore most important that the edges of a slab be made of sufficient thickness so that the bending resistance along the edge will not be exceeded. This applies to all edges, whether they be along the side, at the center joint, or at a transverse joint.

If there is a center joint with no means for transferring load across the joint, the edge along the center joint is stressed to about the same amount as the outside edge of the pavement and should be of the same thickness. When an effective means is provided, however, for transferring load (such as tongue-and-groove construction), the stress along the center joint is considerably decreased.

High tension is produced in the top of the slab near a corner. This tension occurs with a heavy wheel load at the corner. It is, however, considerably less than the longitudinal tension along the bottom outside edge of the slab directly under a heavy wheel load.

A concrete slab should have a thickened edge in order to receive uniformly distributed maximum stress under traffic. This conclusion bears out the conclusions which have been reached as a result of the Pittsburgh Test Road and of the Bates Test Road.

When a center longitudinal joint is provided with an efficient means for transferring load across the joint, the thickness of the slab at the joint should be equal to not less than 0.7 of the thickness at the outside edge in order that like maximum stresses will be received in these two positions.

It is quite strongly indicated that any road roughness or truck conditions tending to produce impact will likewise greatly increase the fiber deformation and the possible likelihood of cracking on this account.

[3] Static load tests on pavement slabs, by J. T. Thompson, PUBLIC ROADS, November, 1924, vol. 5, No. 9, p. 1.

It is quite evident that maximum stresses under the edge of the slab are produced only with the truck wheel trailing along that edge, and that these stresses are very greatly reduced when the truck wheels trail along a line no more than 1 foot distant from the edge. This suggests that where curbs are used which prevent the traffic from reaching the extreme edges, thickened edges probably are not necessary.

INDICATIONS FROM COLUMBIA PIKE EXPERIMENTS

The Columbia Pike investigations have been described in the October, 1924, issue of PUBLIC ROADS.[4] Briefly the road consisted of 32 sections of both plain and reinforced concrete. The variations in the sections consisted of changes in thickness, changes in cross-section design, the use of cinders under some of the sections and the use of various percentages of reinforcing steel.

Careful observations were made of the development of cracking in the sections and subgrade samples were taken from time to time to note the moisture content and also physical characteristics of materials under various sections of the road.

An analysis of the subgrade samples and of the physical condition of the pavement as revealed by the record of longitudinal cracking indicates that the subgrade has a decided influence on the condition of the pavement. Materials subject to high volumetric change and having high "moisture equivalents" generally produce the most extensive longitudinal cracking. The most striking observations from these tests, however, have to do with the behavior of the various sections as influenced by the amount of longitudinal reinforcing steel.

In sections which have from eight to fourteen ¾-inch round deformed rods placed lognitudinally, transverse cracks have occurred at frequent intervals a few feet apart. These cracks, however, are very fine. Sections which have eight ¾-inch rods have not cracked as frequently as those having fourteen ¾-inch rods. One section having four 1-inch rods has cracked slightly less than sections having eight ¾-inch rods. Sections which have reinforcing varying from four ¾-inch to four ½-inch rods are free from fine transverse cracks but have wide, open cracks similar to those found in plain concrete sections but spaced at greater distances—60 to 70 feet—as compared with 10 to 50 feet in plain concrete slabs. Sections which are reinforced with 25 and 50 pounds of mesh per 100 square feet have no fine cracks, but do have open cracks spaced at greater distances than would ordinarily be found in plain concrete. In order to discover the cause for the action of these several sections, the cracks were opened in a number of instances, thus exposing the steel.

It was found that in those sections having eight or more ¾-inch deformed round bars, no change in diameter of the bars had occurred. In the section reinforced with four 1-inch bars, slight reductions in diameter were found. In the section having four ¾-inch bars, the diameter was reduced to ⅝ inch. In the sections having eight ½-inch rods the diameter has been reduced to approximately ⅜ inch, while in still another section having only four ½-inch bars the bars had completely broken with a reduction in diameter to about ¼ inch. In sections reinforced with mesh to the extent of 25 and 50 pounds per 100 square feet, the mesh was completely broken at the cracks. The indications are very strong therefore:

(1) That longitudinal steel up to a certain amount does increase the spacing of transverse cracks and these cracks will open on contraction.

(2) That longitudinal steel above this amount might produce fine transverse cracks, spaced in some cases only a few feet apart.

TEST OF SKEW ARCHES

In the past there have been failures of skew arches generally through excessive stress at the obtuse angle end of the abutment and there are a number of instances of skew arches now constructed which are showing signs of distress at this location. Bridge engineers are in a quandary as to just what constitutes a proper basis for the design of arches in which the abutments are inclined at an angle with the axis of the roadway. A series of tests which is still under way at the Bureau of Public Roads is showing some very striking indications of the wide variation in abutment reactions, both horizontal and vertical, depending upon the angle of skew. Thus far arches have been tested with 60°, 45°, and 30° angles of skew and a 15° arch is now under load. The arches are 7 feet in span, measured on the center line of the roadway, the ring is 4½ feet wide at right angles to the center line of the roadway and the rise is one-fifth of the span.

The principal object sought is the determination of the distribution of abutment reactions, both horizontal and vertical, from one end of the abutment to the other. The results obtained on the three arches tested are very clear in showing that by far the greater amount of the load is transmitted to the obtuse angle end of the abutment. In the case of the 60° skew both the vertical and horizontal reactions at the acute angle end are practically zero and there is a straight-line distribution of vertical reaction from the acute angle to the obtuse angle end in all three arches. The more nearly the angle of the arch approaches a right angle, the more nearly uniform does the distribution of both vertical and horizontal reactions become. It would be premature to draw definite conclusions from these tests at the present time but the indications are quite strong that for arches of these proportions the centroid of the vertical and horizontal reactions for a 60° skew is located one-third of the length of the abutment measured from the obtuse angle, and that its location shifts between that point and the center as the angle of skew decreases. The above is a rather rough statement of the indications but a more detailed statement will be forthcoming shortly in a technical paper covering these investigations.

IMPACT EXPERIMENTS ON BRIDGES

Cooperative impact tests are now being conducted jointly between the Bureau of Public Roads and Iowa State College.[5] Thus far the principal conclusion to be reached is that:

[4] Reinforcing and the subgrade as factors in the design of concrete pavements, by J. T. Pauls, PUBLIC ROADS, October 1924, vol. 5, No. 8, p. 1.

[5] Impact tests on highway bridges, PUBLIC ROADS, September, 1924, vol. 5, No. 7,

> The accurate determination of impact stresses is now possible with the use of several different types of instruments. It is also possible to measure the force of the blow delivered by motor trucks as influenced by a number of factors, such as bridge floor roughness, deflection, cushioning, etc.

A number of impact stresses have been measured but definite conclusions are hardly forthcoming at the present time.

EARTH PRESSURES ON BRIDGE ABUTMENTS OR RETAINING WALLS

The various State highway departments make a wide range of assumptions with regard to earth pressures existing behind retaining walls. A very interesting series of tests has recently been completed in which soil pressure measuring cells have been embedded back of retaining walls during their construction and later, after the fill has been completed measurements of pressure have been obtained. These measurements are quite illuminating and they surely point to the necessity for recognizing the existence of higher earth pressures than are generally thought to exist. One series of tests includes the measurement of pressures back of a bridge abutment some 25 feet in height. The adjacent stream rises to a considerable height during the spring of the year and the filling material becomes thoroughly saturated and slowly dries out during the summer. The pressure readings in this particular case show conclusively that the pressures increase and decrease in accordance with the percentage of moisture in the filling material which in this case was a clay-loam soil. The pressure varies from that which would be caused by a fluid weighing 44.1 pounds per cubic foot to that due to a fluid weighing 29.8 pounds per cubic foot. It is interesting to note that Rankin's formula for pressure under the assumption of earth weighing 100 pounds per cubic foot with an angle of repose of 1½ to 1 gives a pressure equivalent to that of a fluid weighing 28.7 pounds per cubic foot.

Other tests made on bridge abutments in Washington, D. C., show correspondingly high pressures and the pressure varies with the manner of filling. It is indicated very strongly:

> That provision for draining the fill should be one of the major features of the work which should be carefully designed and executed in the construction of the wall or abutment and that the earth pressure should not be taken at less than that which would be developed by a fluid weighing 28 to 30 pounds per cubic foot.

INVESTIGATIONS DEALING WITH MATERIALS OF CONSTRUCTION

Among the outstanding investigations made during the past year, from the standpoint of its possible economic importance, is the study of the bulking effect of moisture in sand.

Concrete is ordinarily proportioned with the thought that the proportions used will give a certain minimum desired strength and certain desired qualities of workability and density. The general ideas which engineers now have with regard to the strength of concrete are based largely on laboratory tests in which the sand is invariably measured in an air-dry condition. In the field the sand is generally moist and is therefore bulked depending upon its moisture and fineness up to 25 per cent of its air-dry volume. There is little wonder then, that concrete proportioned by volume with moist sand does not have the same characteristics as concrete proportioned in the manner ordinarily used in the laboratory nor need there be much surprise that field-proportioned concrete, in general, is harsher and does not possess the workability of concrete of the same nominal proportions as made in the laboratory, and, further, it is quite plain why the yield of field concrete is sometimes less than is anticipated.

The bulking tests made during the past year at least point to the desirability of basing the proportions of concrete on the volume of sand as measured in its dry condition. The proportions would then have a much more definite meaning than they have at present, the sand now being measured without regard to its moisture content and condition of bulking. More study should be given to this subject. It has considerable significance. It is interesting to note that proportioning by weight as now practiced in Iowa very largely eliminates the difficulty due to the bulking of sand, but other methods also may be employed.

INVESTIGATIONS OF THE CHARACTERISTICS OF AGGREGATES SUITABLE FOR CONCRETE PAVEMENTS

One rather extensive investigation, completed during the past year at Arlington, Va., was made primarily for determining the limit of characteristics of aggregates suitable for use in concrete pavement construction. The main question involved was the determination of the minimum standard of quality. To answer this question a number of aggregates were imported from different sections of the country and a concrete-surfaced circular track was constructed with these materials. This track was tested by a special-wear machine, first fitted with rubber tires alone then equipped with tire chains.

The conclusions which have been drawn as a result of these tests are based on three premises: [6]

1. That on the average 18-foot concrete road, approximately 10 per cent of the total traffic moving in both directions passes over a band 6 inches in width at the point of maximum concentration. Therefore, the traffic passing over the test road should be multiplied by at least 10 to obtain the equivalent volume of traffic on an actual pavement.

2. That the comparative resistance of each test section to the surface wear produced by the tire chains is an indication of the comparative resistance which would be offered by the concrete to wear or disintegration at the edges of exposed joints and cracks under service conditions.

3. That conditions which cause surface wear, such as steel tires, tire chains tracking through snow, etc., are present to such an extent as to make it necessary to give the question of wear consideration in the selection of concrete aggregates.

From the results obtained the following conclusions were drawn:

> That the rate of wear of stone concrete is, in general, not affected by the coarse aggregate, provided the coarse aggregate is equal or superior to the mortar matrix in resistance to wear.

[6] Wear of concrete pavement tested, by F. H. Jackson and J. T Pauls, PUBLIC ROADS, May, 1924, vol. 5, No. 3, p. 1.

That excessive wear will result from the use of very soft stone as coarse aggregate even though used in conjuction with a mortar of satisfactory quality. From the results of these comparative tests, it would appear that stone with a percentage of wear over seven should not be used in concrete road construction.

That gravel concrete, in general, is at least as satisfactory from the standpoint of wear as stone concrete.

That gravels consisting essentially of siliceous materials are superior as regards both the amount and uniformity of wear to those containing a preponderance of calcareous fragments.

That gravels consisting of rounded particles are as satisfactory from the standpoint of wear as those consisting either wholly or in part of angular or crushed fragments.

That small amounts of shale occurring in the coarse aggregate will cause both excessive and uneven wear.

That the modified abrasion test for gravel in its present form is not an indication of the wear-resisting properties of coarse aggregates. It is suggested that if the severe impact action of the steel balls were decreased, much more indicative results would be secured.

The blast furnace slags should prove satisfactory for use in concrete pavements provided the proportion of light, porous slag is so controlled that the weight per cubic foot will be at least 70 pounds.

That the presence of large amounts of light, porous fragments in blast furnace slag will cause excessive wear.

That somewhat better results are secured by the use of the smaller sizes of slag.

That slag or stone screenings are, in general, unsatisfactory as substitutes for natural sand as fine aggregate in concrete road construction.

That the copper and lead smelter slags used in these tests would make satisfactory aggregates for concrete road construction from the standpoint of wear.

That coarse sands, other things being equal, show greater resistance to wear than fine sands.

That the so-called "tensile-strength-ratio" test is no indication of the wear-resisting properties of concrete made with these sands.

That the Talbot-Jones wear test is not, in general, an indication of the wear which takes place under traffic.

That neither the crushing nor the transverse strength of concrete is a measure of its wear-resisting properties.

That the addition of hydrated lime in the proportion used in these tests does not affect the wear-resisting properties of concrete.

That so far as resistance to wear alone is concerned, increasing the cement content beyond a cement-sand ratio of 1 : 2 does not materially affect the concrete. Leaner mixes on the other hand show marked increase in wear.

That unusual precautions should be taken in using mine chats or other similar harsh-working materials, so as to increase workability to a maximum and thus make possible a smoother surface finish.

That, other things being equal, either an excessively dry or an excessively wet mix will show less resistance to wear than concrete of medium consistency.

A number of other tests are under way covering a wide variety of subjects having to do with both materials and design. From these researches indications and definite conclusions are gradually being drawn. All of these investigations should be of interest to the highway engineer. They involve such tests as fatigue of concrete, the effect of calcium chloride curing on concrete, investigations of bituminous materials, the study of the laws governing the stability of bituminous mixtures, causes of scaling of concrete roads, and a large number of other problems, each having an important bearing on the production of better and more economical pavements and structures.

COMPARISONS OF THE STRENGTH OF CONCRETE IN TENSION AND COMPRESSION

By N. M. FINKBINER, Engineer of Materials, Oregon State Highway Commission

During the 1922 construction season, the materials department of the Oregon State Highway Commission decided to run a few tension tests on concrete, to see if a general relation exists between the tensile and compressive stength. Some of the specimens were cast in the laboratory using a variety of materials, and some were cast in the field on the various concrete paving jobs then under construction. The laboratory specimens, both tension and compression, were cured in moist sand. In the field both classes of specimens were cured on top of the pavement, receiving the same conditions of curing as the pavement itself, which is standard Oregon practice.

The forms in which the tension specimens were cast, were shaped like a standard cement briquette, but were 36 square inches in cross section at the narrow part, the dimensions being 6 by 6 inches. The compression specimens were cast in the regular 6 by 12 inch sheet metal molds. Jaws which proved very satisfactory were designed to be used on a universal testing machine.

The specimens cast in the field gave as an average 288 pounds to the square inch in tension, and 3,185 pounds to the square inch compression. This indicates that the compression is about 11.06 times the tension. The specimens cast in the laboratory gave the compression as 8.9 times the tension. These averaged 241 pounds to the square inch tension, and 2,160 pounds to the square inch compression. The laboratory concrete specimens were expected to show a discrepancy, should any exist, on account of the fact that while the mixes were all 1: 2: 3, and the consistencies were kept as closely as practicable the same, different mineral aggregates were used, some of which would not pass specifications, nor be permitted to be used in State work. Under general field conditions in Oregon, therefore, the compression in pounds per square inch divided by 10 will give approximately the strength of the concrete in tension at the age of 28 days.

HIGHWAY INCOME FROM THE MOTOR VEHICLE

By HENRY R. TRUMBOWER, Economist, United States Bureau of Public Roads

COMPARING the two principal means of transportation in the United States, namely the railroads and highways, certain similarities and differences are noted. The railroad transportation system of the country is wholly in private hands. The roadbed and tracks and also the equipment are owned by organizations of private investors. With respect to highway transportation, on the other hand, we find the situation quite different. The equipment, i. e., the motor vehicles, both passenger automobiles and motor trucks, the storage houses and everything that pertains to the operation of the cars and trucks is owned by individuals or by private organizations. The roadbed itself is owned, constructed, and maintained by the public, except in a very few instances where toll roads and toll bridges still exist. In railroad transportation the rate which one pays for transporting passengers or freight is intended to cover the costs relating to the operation of the equipment and the maintenance of the roadbed and track and also the interest on the value of the property used. In highway transportation we find that the same costs are present which prevail with reference to railroad transportation. The cost of the automobile or the motor truck and its operating costs have to be met, and the cost of constructing and maintaining the highway has to be paid by some one. Under the system which prevails at the present time the operator and owner of the motor vehicle bears its operating costs and the public, through State highway commissions or through local highway organizations, bears the costs of constructing and maintaining the highway. One of the main problems of highway finance is to apportion this highway cost among those who, on the one hand, make a direct use of the highway and benefit thereby, and those, on the other hand, who are benefited indirectly through the development of the highway system. Leaving out of consideration the toll roads, which formerly were fairly numerous, but now have been largely taken over by the public, the public pays for the construction and maintenance of the highways. Formerly the funds necessary for this purpose were raised by general property taxes or, in some cases, from assessments levied on land directly adjacent to or close to the improved highway. To these sources of income there has been added since the advent of the motor vehicle and the growth of a demand by the owners of such vehicles for more extensive highway improvement, another source, which is the tax laid upon the owners of the vehicles, or, in other words, the road users.

These motor-vehicle revenues are raised mainly by two methods. First, by the licensing of the motor vehicles and the exaction of a special fee in connection therewith; and, second, by the taxation of gasoline, which provides revenues proportionate to the use made

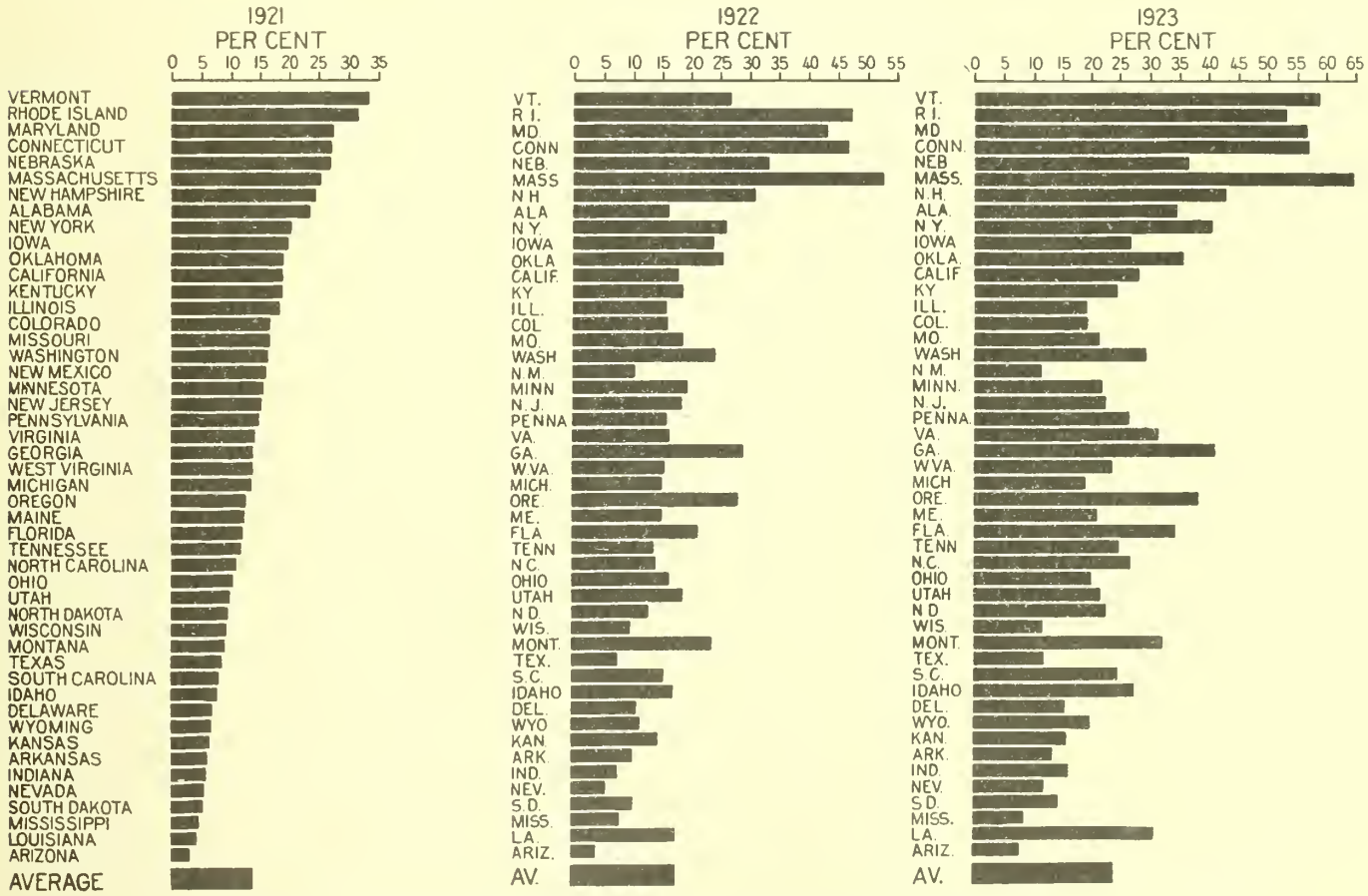

FIG. 1.—Ratio of total motor vehicle license fees and gasoline taxes to total rural highway expenditures exclusive of interest and principal payments

of the streets and roads. Through the use of these two methods the automobile is called upon to bear directly a part of the total cost of highway construction and maintenance.

MOTOR VEHICLE REVENUES COMPARED WITH TOTAL HIGHWAY EXPENDITURES

In Tables 1, 2, and 3 and Figure 1 are shown by States for the years 1921, 1922, and 1923, respectively, the total highway expenditures and the total revenues derived from motor-vehicle license fees and from the gasoline taxes and also the ratios which these total motor vehicle revenues bore to the total highway expenditures. The expenditures as tabulated here for the year 1921 cover the items of construction and maintenance of roads and bridges, administration and engineering, and the purchase and repair of machinery and equipment and general and miscellaneous expenses. Payments made as interest and principal payments on highway bonds are excluded. For 1922 the total construction and maintenance items were obtained directly from State reports, and the items of purchase and repair of equipment and general and miscellaneous expense were estimated as bearing the same ratio to the construction and maintenance items as in the previous year. For 1923 no definite expenditure figures for the individual States are available. However, it is known that the expenditures made by the State highway departments during the year were practically the same as in 1922, and there is strong indication that the total 1923 expenditures were practically the same as the 1922 expenditures for the country as a whole, although there may have been variations for individual States which these estimates do not take into account. The percentages show at least the trend of the ratio between motor-vehicle revenues and highway expenditures. In Figure 1 the ratios which the motor-vehicle revenues bear to the total highway expenditures for these three years are compared, the States being arranged in the order of the 1921 ratios.

In 1921 the total highway expenditures of the country, exclusive of interest and principal payments, amounted to $947,306,826, and the total license fees and gasoline taxes, exclusive of the District of Columbia, were $127,571,331, which was 13.4 per cent of the total highway expenditures. The total highway expenditures for 1922 were $931,886,835; and in that year the total motor vehicle-revenues amounted to $160,854,384, or 17.2 per cent of the highway expenditures. Using the same expenditure figures for 1923 as for 1922, we find that the total motor-

TABLE 1.—*Total highway expenditures and motor vehicle revenues, 1921*

State	Total highway expenditures, less interest and prineipal payments	Total license fees and gasoline taxes	
		Amount	Pereentage of total highway expenditures
			Per cent
Alabama	$4,881,701	$1,147,265	23.5
Arizona	9,804,812	283,898	2.9
Arkansas	19,353,806	1,026,544	5.8
California	36,614,695	6,834,090	18.7
Colorado	8,903,278	1,465,532	16.5
Connecticut	8,445,716	2,306,350	27.2
Delaware	5,570,049	375,469	6.7
Florida	8,541,667	1,018,712	11.8
Georgia	14,571,511	2,008,099	13.7
Idaho	10,786,437	841,213	7.8
Illinois	37,639,731	6,803,556	18.1
Indiana	44,142,148	2,422,227	5.5
Iowa	39,324,553	7,719,128	19.6
Kansas	22,054,780	1,400,000	6.4
Kentucky	11,683,078	2,183,826	18.7
Louisiana	11,838,160	453,276	3.8
Maine	8,259,725	1,004,750	12.1
Maryland	8,968,584	2,460,162	27.4
Massachusetts	18,634,337	4,717,389	25.3
Michigan	50,708,494	6,751,925	13.3
Minnesota	37,144,902	5,672,424	15.3
Mississippi	17,256,456	751,946	4.3
Missouri	15,240,889	2,505,354	16.4
Montana	9,276,916	823,320	8.9
Nebraska	10,361,131	2,824,811	27.0
Nevada	1,971,895	102,800	5.2
New Hampshire	3,598,921	876,322	24.4
New Jersey	26,334,950	3,974,064	15.0
New Meixeo	3,369,464	531,921	15.8
New York	50,913,742	10,288,858	20.2
North Carolina	25,617,735	2,765,258	10.8
North Dakota	7,247,231	683,053	9.5
Ohio	65,777,680	6,894,160	10.2
Oklahoma	13,931,478	2,619,714	18.8
Oregon	26,476,121	3,270,058	12.4
Pennsylvania	69,580,813	10,305,499	14.8
Rhode Island	2,693,534	848,724	31.5
South Carolina	9,444,868	741,115	7.9
South Dakota	13,752,165	720,587	5.2
Tennessee	12,046,793	1,387,870	11.5
Texas	45,715,452	3,806,395	8.3
Utah	4,564,239	441,360	9.7
Vermont	2,150,484	668,289	33.1
Virginia	14,383,422	2,021,146	14.0
Washington	22,229,050	3,612,578	16.2
West Virginia	9,110,841	1,250,526	13.7
Wiseonsin	40,774,180	3,671,646	9.0
Wyoming	4,329,212	288,122	6.6
	947,306,826	127,571,331	13.4

TABLE 2.—*Total highway expenditures and motor-vehicle revenues, 1922*

State	Total highway expenditures, less interest and prineipal payments	Total lieense fees, and gasoline taxes	
		Amount	Pereentage of total highway expenditures
			Pe reent
Alabama	$7,771,268	$1,262,800	16.2
Arizona	9,562,166	374,468	3.9
Arkansas	12,292,781	1,238,271	10.0
California	46,886,633	8,384,606	17.9
Colorado	10,334,618	1,636,542	15.9
Connectieut	9,118,682	4,256,991	46.6
Delaware	3,944,090	426,377	10.8
Florida	10,549,971	2,231,563	21.2
Georgia	8,878,320	2,569,235	29.0
Idaho	4,784,041	812,943	17.0
Illinois	50,496,350	7,882,482	15.6
Indiana	40,689,112	2,999,588	7.4
Iowa	33,401,849	7,923,388	23.7
Kansas	21,709,498	3,100,000	14.3
Kentucky	13,884,050	2,587,993	18.6
Louisiana	12,786,192	2,240,618	17.5
Maine	9,467,482	1,417,507	15.0
Maryland	7,497,713	3,220,387	43.0
Massaehusetts	10,843,800	5,685,527	52.6
Michigan	55,516,403	8,305,022	15.0
Minnesota	33,644,891	6,543,685	19.5
Mississippi	18,078,341	1,444,542	8.0
Missouri	18,913,961	3,512,182	18.6
Montana	3,635,170	863,811	23.7
Nebraska	9,134,304	3,031,699	33.2
Nevada	2,240,623	120,937	5.4
New Hampshire	4,047,980	1,246,098	30.9
New Jersey	34,195,623	6,251,418	18.3
New Mexico	4,159,433	426,901	10.2
New York	48,952,729	12,736,364	26.0
North Carolina	24,949,161	3,493,827	14.0
North Dakota	5,417,705	698,931	12.9
Ohio	48,234,644	7,888,992	16.3
Oklahoma	10,721,964	2,729,169	25.5
Oregon	15,851,436	4,440,779	28.0
Pennsylvania	80,699,582	12,575,380	15.6
Rhode Island	2,414,704	1,139,742	47.2
South Carolina	9,810,758	1,501,888	15.3
South Dakota	12,116,778	1,232,232	10.2
Tennessee	11,659,311	1,592,230	13.7
Texas	56,022,344	4,261,488	7.6
Utah	3,909,295	729,455	18.7
Vermont	2,882,200	781,982	27.0
Virginia	15,143,391	2,467,346	16.2
Washington	17,564,039	4,245,500	24.1
West Virginia	12,546,208	1,936,079	15.4
Wisconsin	41,706,869	4,088,570	9.8
Wyoming	2,818,372	316,849	11.2
	931,886,835	160,854,384	17.2

vehicle revenues of $225,427,013, again exclusive of the District of Columbia, amounted to 24.2 per cent of the amount of money spent for highway construction and maintenance. In 1924 it is believed that the total motor-vehicle revenues from license fees and gasoline taxes will approximate $300,000,000, which will be about one-third of the total highway expenditures assuming that the 1924 expenditures will not be greatly different from those of preceding years.

During this four-year period the increase in motor-vehicle revenues was about 135 per cent of the 1921 amount, while the motor-vehicle registration increased but 50 per cent. The increase in revenues was due not only to the larger number of cars registered but also to the raising of the license fees on passenger cars and the further increased fees on motor trucks and to the added revenues produced by the gasoline tax.

In 1923 there were five States—Massachusetts, Vermont, Connecticut, Maryland, and Rhode Island—in which the motor-vehicle revenues were more than 50 per cent of the total highway expenditures for the year. In Massachusetts they amounted to 64.5 per cent, and this State was the only one in which the ratio was over 70 per cent in 1922. The five States which raised the smallest part of the total funds expended by motor-vehicle taxation in 1923 were Arizona, Mississippi, New Mexico, Texas, and Wisconsin, in which States the ratio between motor-vehicle revenues and highway expenditures was less than 12 per cent. It will be observed also that in 1923 there were 26 States in which motor-vehicle revenues were less than 25 per cent of the total highway expenditures and 22 in which they were greater, whereas in 1921 there were only six States—Connecticut, Maryland, Massachusetts, Nebraska, Rhode Island, and Vermont—in which the motor-vehicle revenues equaled or exceeded 25 per cent of the highway expenditures. The trend during the three-year period will be clear from these comparisons.

TABLE 3.—*Estimated total highway expenditures and motor-vehicle revenues, 1923*

State	Total highway expenditures, less interest and principal payments	Total license fees and gasoline taxes	
		Amount	Percentage of total highway expenditures
			Per cent
Alabama	$7,771,268	$2,674,103	34.4
Arizona	9,562,166	755,794	7.9
Arkansas	12,292,781	1,654,289	13.5
California	46,886,633	13,127,437	28.0
Colorado	10,334,618	1,972,572	19.1
Connecticut	9,118,682	5,209,655	57.0
Delaware	3,944,090	604,788	15.3
Florida	10,549,971	3,604,108	34.1
Georgia	8,878,320	3,658,910	41.2
Idaho	4,784,041	1,310,502	27.4
Illinois	50,496,350	9,653,796	19.1
Indiana	40,689,112	6,600,143	16.2
Iowa	33,401,849	8,827,063	26.5
Kansas	21,709,498	3,435,606	15.8
Kentucky	13,884,050	3,359,168	24.2
Louisiana	12,786,192	3,945,679	30.9
Maine	9,467,482	1,946,345	20.8
Maryland	7,497,713	4,225,259	56.2
Massachusetts	10,843,800	6,989,633	64.5
Michigan	55,516,403	10,500,786	18.9
Minnesota	33,644,891	7,316,772	21.8
Mississippi	18,078,341	1,545,472	8.5
Missouri	18,913,961	4,016,384	21.2
Montana	3,635,170	1,170,871	32.2
Nebraska	9,134,304	3,353,175	36.6
Nevada	2,240,623	269,731	12.0
New Hampshire	4,047,980	1,734,391	42.9
New Jersey	34,195,623	7,653,780	22.4
New Mexico	4,159,433	460,000	11.1
New York	48,952,725	19,862,442	40.6
North Carolina	24,949,161	6,637,949	26.6
North Dakota	5,417,705	1,221,934	22.5
Ohio	48,234,644	9,662,370	20.0
Oklahoma	10,721,964	3,816,771	35.5
Oregon	15,851,436	6,027,751	38.2
Pennsylvania	80,699,582	21,335,826	26.4
Rhode Island	2,414,704	1,286,659	53.2
South Carolina	9,810,758	2,414,061	24.5
South Dakota	12,116,778	1,755,652	14.5
Tennessee	11,659,311	2,862,010	24.6
Texas	56,022,344	6,657,132	11.9
Utah	3,909,295	834,191	21.4
Vermont	2,882,200	1,107,033	58.8
Virginia	15,143,391	4,757,083	31.5
Washington	17,564,039	5,123,747	29.2
West Virginia	12,546,208	2,974,998	23.7
Wisconsin	41,706,869	4,958,934	11.8
Wyoming	2,818,372	554,258	19.7
	931,886,835	225,427,013	24.2

The relation between motor vehicle revenues and highway expenditures having been established, it now remains to be seen in what manner the motor vehicle revenues are raised. Tables 4 and 5 and Figure 2 show, for the years 1921 and 1923, respectively, the average motor vehicle revenues per vehicle and the division of these revenues between license fees and gasoline taxes. In 1921 the license fees collected by the several States amounted to $122,269,072, covering a total registration of 10,422,670 motor vehicles, from which it appears that the average payment was $11.70

TABLE 4.—*Average license fees and gasoline taxes, 1921*

State	Average motor vehicle revenues per vehicle	Average license fees per vehicle		Average gasoline tax per vehicle	
		Amount	Percentage of motor vehicle revenues	Amount	Percentage of motor vehicle revenues
					Per cent
Alabama	$13.90	$13.90	100		
Arizona	10.85	5.50	51	$5.35	49
Arkansas	15.20	12.70	89	2.50	11
California	10.00	10.00	100		
Colorado	10.00	6.20	62	3.80	38
Connecticut	17.21	15.90	92	1.31	8
Delaware	17.50	17.50	100		
Florida	10.40	7.50	72	2.90	28
Georgia	15.20	12.90	85	2.30	15
Idaho	16.40	16.40	100		
Illinois	10.20	10.20	100		
Indiana	6.10	6.10	100		
Iowa	16.70	16.70	100		
Kansas	4.85	4.85	100		
Kentucky	17.25	14.00	81	3.25	19
Louisiana	5.80	5.80	100		
Maine	12.90	12.90	100		
Maryland	18.10	18.10	100		
Massachusetts	13.10	13.10	100		
Michigan	14.20	14.20	100		
Minnesota	17.60	17.60	100		
Mississippi	11.50	11.50	100		
Missouri	7.20	7.20	100		
Montana	14.00	10.10	72	3.90	28
Nebraska	11.90	11.90	100		
Nevada	9.50	9.50	100		
New Hampshire	20.80	20.80	100		
New Jersey	14.50	14.50	100		
New Mexico	23.50	8.80	37	14.70	63
New York	12.70	12.70	100		
North Carolina	18.60	15.20	82	3.40	18
North Dakota	7.40	7.40	100		
Ohio	9.60	9.60	100		
Oklahoma	11.80	11.80	100		
Oregon	27.60	19.70	71	7.90	29
Pennsylvania	14.90	13.70	92	1.20	8
Rhode Island	15.50	15.50	100		
South Carolina	8.20	8.20	100		
South Dakota	6.10	6.10	100		
Tennessee	11.80	11.80	100		
Texas	8.10	8.10	100		
Utah	9.30	9.30	100		
Vermont	17.90	17.90	100		
Virginia	14.50	14.50	100		
Washington	19.50	17.00	87	2.50	13
West Virginia	13.30	13.30	100		
Wisconsin	10.80	10.80	100		
Wyoming	10.70	10.70	100		
Average, States having gasoline tax	16.14	13.40	83	2.74	17
Average, all States	12.20	11.70	96	.50	4

COMPARISON OF AVERAGE LICENSE FEES AND GASOLINE TAXES, 1921 AND 1923

per vehicle. In the same year 13 States collected $5,302,259 in gasoline taxes, which amounted to an average of 50 cents per motor vehicle for the country as a whole, or $2.74 per motor vehicle for those States in which the gasoline tax was in effect. If we divide the sum of all the license fees and the gasoline taxes by the total number of motor vehicles, we get an average of $12.20 paid by each vehicle. For those States which had the gasoline tax the average motor vehicle revenues were $16.14 per vehicle. The highest average payment per motor vehicle was in Oregon, where 71 per cent of the average payment of $27.60 was derived from license fees and 29 per cent from gasoline taxes. Kansas collected the lowest average revenue, or $4.85 per vehicle, which was wholly in license fees. In the same year the average motor vehicle revenue exceeded $20 in but three States—New Hampshire, New Mexico, and Oregon; in 34 States it was between $10 and $19, and in 11 States, less than $10.

In 1923, the last year for which full and complete data are available, the total license fees and permits, exclusive of the District of Columbia, amounted to $188,613.074 for a registration of 15,017,366 motor vehicles, or an average of $12.52 per vehicle. A gasoline tax was collected in 35 States, yielding a total of $36,813,939, which was an average payment of $4.90 for those States in which the tax was in effect. If the gasoline tax revenues are applied to the total registration of the country, the average payment per vehicle is reduced to $2.44. The total motor vehicle revenues of all the States represented an average of $14.96 per vehicle; if the 35 States which had in effect a gasoline tax are segregated, the motor vehicle revenues in those States averaged $16.60 per vehicle. In this year there were 15 States in which the motor vehicle revenues averaged $20 and over per vehicle; in 28 States the average ranged between $10 and $19; and in 5 States it was less than $10 per vehicle.

The average motor vehicle revenues for 1921 and 1923 can be compared thus:

	1921	1923
States in which average revenues were $20 and over	3	15
States in which average revenues were $10 to $19	34	28
States in which average revenues were $10 and less	11	5

GASOLINE CONSUMPTION AND TAX RECEIPTS PER CAR FOR PERIOD, JANUARY 1 TO JULY 1, 1924

In the States which collected a gasoline tax during the first six months of 1924 the collections amounted to $4.18 per car (Table 6, fig. 3). The average consumption per car was 211.5 gallons. It is estimated that the year's consumption will average more than twice this amount because there will be more auto-

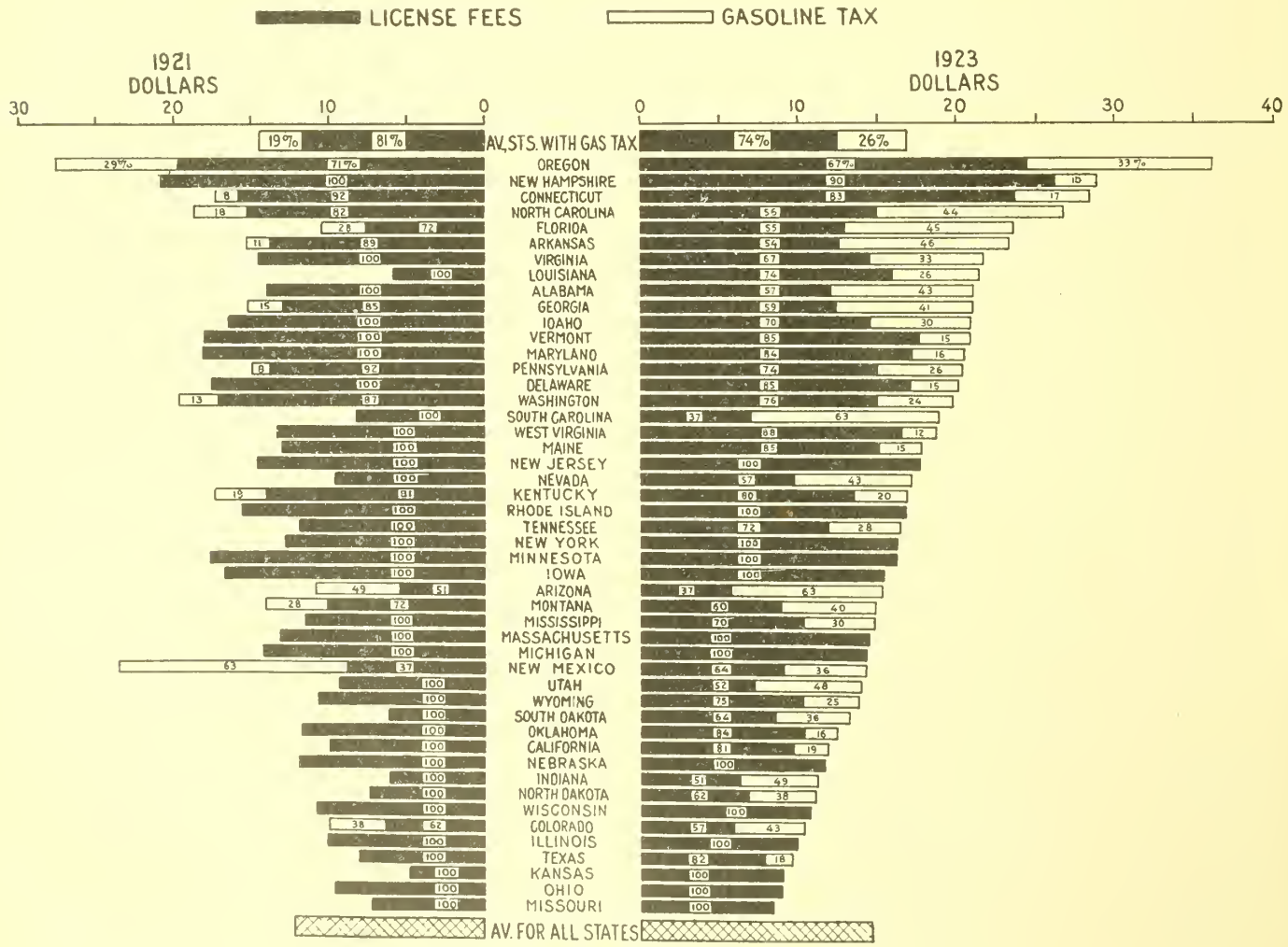

FIG. 2.—Average revenue per motor vehicle, 1921 and 1923

TABLE 5.—*Average license fees and gasoline taxes, 1923*

State	Average motor-vehicle revenues per vehicle	Average license fees per vehicle		Average gasoline tax per vehicle	
		Amount	Percentage of motor-vehicle revenues	Amount	Percentage of motor-vehicle revenues
			Per cent		*Per cent*
Alabama	$21.12	$12.17	57	$8.95	43
Arizona	15.37	5.73	37	9.64	63
Arkansas	23.43	12.67	54	10.76	46
California	11.93	9.64	81	2.29	9
Colorado	10.44	5.96	57	4.48	43
Connecticut	28.66	23.82	83	4.84	17
Delaware	20.17	17.22	85	2.95	15
Florida	23.72	12.92	55	10.80	45
Georgia	21.04	12.40	59	8.64	41
Idaho	21.00	14.65	70	6.35	30
Illinois	9.94	9.94	100		
Indiana	11.31	6.33	56	4.98	44
Iowa	15.46	15.46	100		
Kansas	9.15	9.15	100		
Kentucky	16.93	13.50	80	3.43	20
Louisiana	21.56	16.04	74	5.52	26
Maine	17.92	15.29	85	2.63	15
Maryland	20.57	17.22	84	3.35	16
Massachusetts	14.53	14.53	100		
Michigan	14.37	14.37	100		
Minnesota	16.33	16.33	100		
Mississippi	14.82	10.33	70	4.49	30
Missouri	8.43	8.43	100		
Montana	14.86	9.88	60	5.98	40
Nebraska	11.72	11.72	100		
Nevada	17.18	9.80	57	7.38	43
New Hampshire	29.10	26.36	90	2.74	10
New Jersey	17.76	17.76	100		
New Mexico	14.36	9.21	64	5.15	36
New York	16.49	16.49	100		
North Carolina	26.89	15.10	56	11.79	44
North Dakota	11.18	6.96	62	4.22	38
Ohio	9.04	9.04	100		
Oklahoma	12.43	10.48	84	1.95	16
Oregon	36.32	24.52	67	11.80	33
Pennsylvania	20.44	15.18	74	5.26	26
Rhode Island	16.86	16.86	100		
South Carolina	18.94	7.08	37	11.86	63
South Dakota	13.33	8.59	64	4.74	36
Tennessee	16.51	11.82	72	4.69	28
Texas	9.68	7.91	82	1.77	18
Utah	14.02	7.23	52	6.79	48
Vermont	20.98	17.79	85	3.19	15
Virginia	21.73	14.62	67	7.11	33
Washington	19.84	15.10	76	4.74	24
West Virginia	18.84	16.52	88	2.32	12
Wisconsin	10.84	10.84	100		
Wyoming	13.92	10.40	75	3.52	25
Average, States having gasoline tax	16.60	11.70	71	4.90	29
Average, all States	14.96	12.52	84	2.44	16

mobile operation in the latter half of the year than in the first half by reason of the better road and weather conditions. So far as can be ascertained, the increasing of the gasoline-tax rate does not retard the sale of gasoline or the consumption per car. An exact comparison in the amount consumed per car per unit of time can not be made for all States because of the rate changes which took place at odd times and because of irregularities in reporting. In the case of Florida and Georgia, however, such a comparison is possible. Both had in effect a 1-cent rate during the first six months of 1923, and the average consumption of gasoline per car during that period was 318 gallons in Florida and 266 gallons in Georgia. With a 3-cent tax in effect during the first six months of 1924 the consumption in Florida was increased to 373 gallons per car, and in Georgia it was increased to 310 gallons per car. In Pennsylvania, South Dakota, and Washington the gasoline tax for the first six months of 1923 was 1 cent, and for the first six months of 1924 it was 2 cents. The average consumption per car for these same periods increased from 155 gallons to 162 gallons in Pennsylvania; from 136 gallons to 187 gallons in South Dakota; and from 178 gallons to 243 gallons in Washington.

TABLE 6.—*Gasoline consumption and gasoline-tax revenues per car, first six months of 1924*

State	Gasoline consumption per car	Gasoline-tax revenues per car
	Gallons	
Alabama	239	$4.78
Arizona	220	6.60
Arkansas	210	8.40
California	253	5.06
Colorado	200	4.00
Connecticut	223	2.23
Delaware	192	3.84
District of Columbia	260	5.20
Florida	373	11.19
Georgia	310	9.30
Idaho	181	3.62
Indiana	181	3.62
Kentucky	163	4.89
Louisiana	285	2.85
Maine	150	1.50
Maryland	158	3.16
Mississippi	152	4.56
Montana	91	1.82
Nevada	145	2.90
New Hampshire	159	3.18
New Mexico	243	2.43
North Carolina	236	7.08
North Dakota	90	.90
Oklahoma	172	4.30
Oregon	225	6.75
Pennsylvania	162	3.24
South Carolina	240	7.20
South Dakota	187	3.74
Tennessee	198	3.96
Texas	266	2.66
Utah	170	4.25
Vermont	114	1.14
Virginia	202	6.06
Washington	243	4.86
West Virginia	144	2.88
Wyoming	201	2.01
Average	211.5	4.18

AVERAGE PASSENGER CAR AND MOTOR TRUCK FEES COMPARED FOR 28 STATES, FIRST SIX MONTHS, 1924

Comparison of passenger car and motor truck fees can be made for 28 States which, for the first six months of 1924, reported separately the license fees derived from passenger cars and from motor trucks and the number of each registered. The average passenger car license fee ranged from $3.17 in California to $28.20 in Idaho (Table 7, Fig. 4). The average for these States was $10.70, though it should be noted that in 17 of the States the average fees were larger than the average license fee for the 28 States as a whole.

The average motor truck license fee for these same States was $21.90, or more than twice as much as the average passenger car fee. The lowest average is found in Montana, where the average motor truck was charged only $11.60 for a license, and the highest is in Oregon, where the average was $51.80. In this case the average was also the mean; 14 of the States charged a license fee greater than the average, and 14 charged less. There is a marked tendency to charge higher license fees for motor trucks than for passenger cars even in those States where the average passenger car license fees are low. In the 11 States where the passenger car license fees are less than the average, or less than $10.70, the motor truck license fees are, on the average, 143 per cent higher; for the 18 States in which the average passenger car fees are above $10.70, the average motor truck fees are only 59 per cent higher.

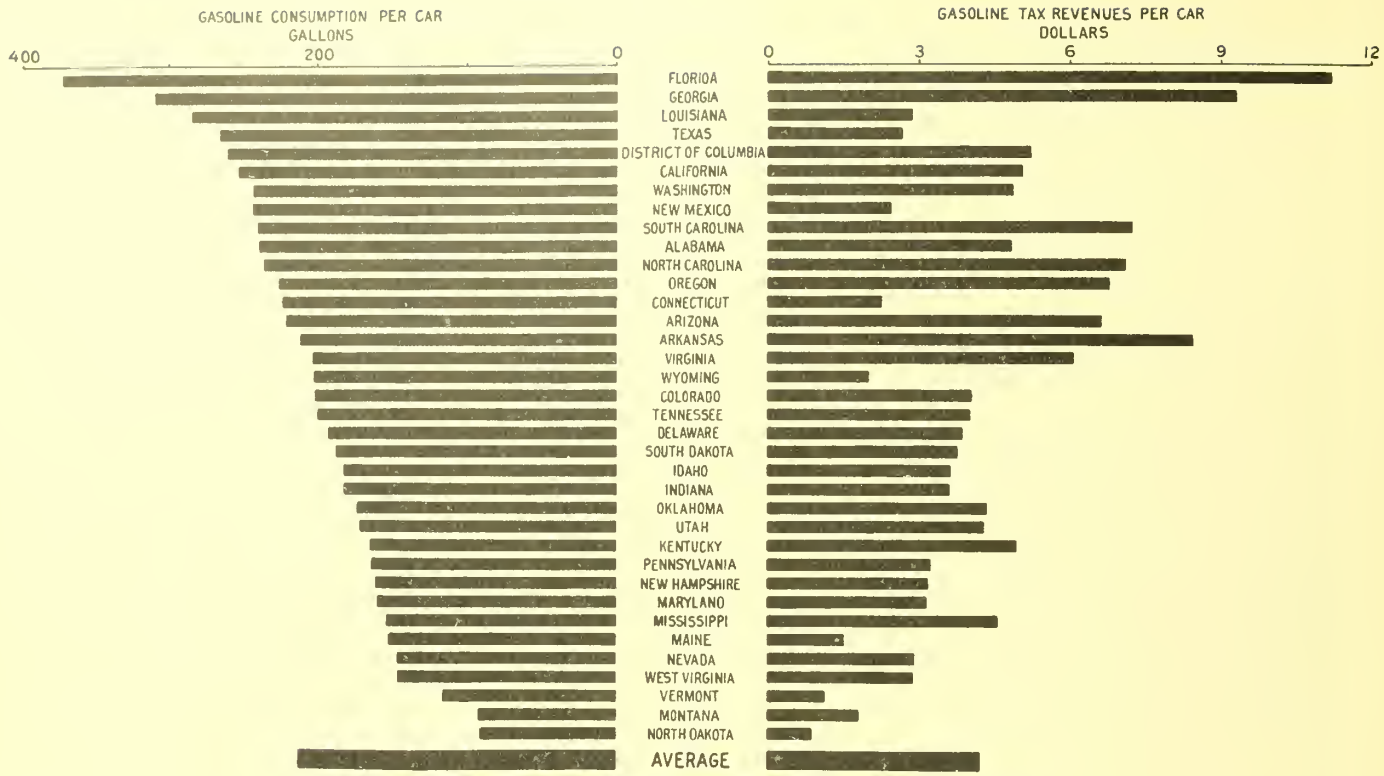

FIG. 3.—Gasoline taxes per car and gasoline consumption per car, first six months of 1924

TABLE 7.—*Average passenger car and motor truck license fees, first six months, 1924*

State	Average passenger car fee	Average motor truck fee
California	$3.17	$13.00
Colorado	5.31	12.35
Connecticut	16.45	34.30
Delaware	11.80	21.20
Georgia	12.25	16.60
Idaho	28.20	27.60
Illinois	8.90	19.25
Indiana	7.15	13.30
Louisiana	16.00	17.25
Maryland	7.55	23.70
Massachusetts	10.55	15.10
Michigan	12.85	17.70
Minnesota	16.20	25.30
Montana	9.48	11.60
Nebraska	11.15	19.50
New Jersey	9.80	28.65
New Mexico	8.80	22.80
New York	12.85	28.30
Oregon	24.10	51.80
Pennsylvania	10.40	29.70
Rhode Island	12.80	21.75
South Carolina	6.95	25.50
South Dakota	14.75	21.30
Vermont	18.70	25.90
Virginia	13.65	16.00
Washington	13.40	24.30
West Virginia	13.10	23.00
Wisconsin	11.80	22.10
Average	10.70	21.90

AMOUNT OF MOTOR VEHICLE FEES AND GASOLINE TAX RECEIPTS APPLICABLE TO HIGHWAY WORK BY OR UNDER SUPERVISION OF STATE HIGHWAY DEPARTMENTS, 1923

Tables 8 and 9 show the gross motor vehicle license fees and gasoline tax receipts and the amounts which the several States devote to highway work under the direction of their State highway departments. The tabulations cover the year 1923. Of the $188,613,074 collected as motor vehicle license fees, $153,226,636, or 81 per cent, was used for highway work by the State highway departments. In 14 States 100 per cent of the license fees is turned over to the State highway departments, in 17 States the State highway department directs the expenditure of from 90 to 99 per cent, in 10 between 50 and 89 per cent, and in 7 States the amount of motor vehicle license fees applicable to highway work by or under supervision of State highway departments is between 10 and 49 per cent of the total collected in this manner.

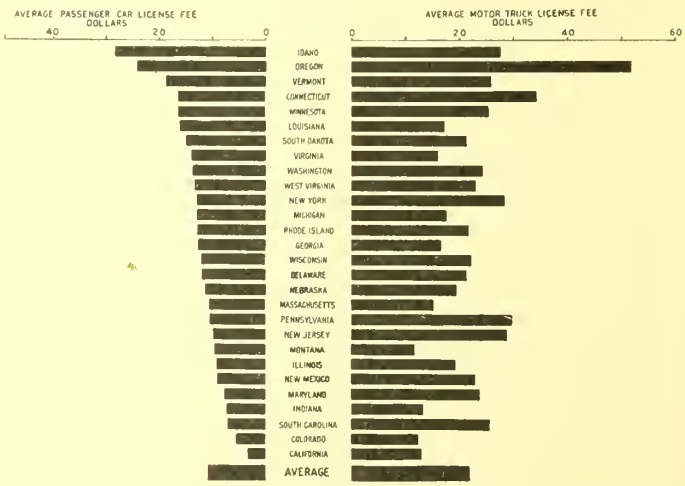

FIG. 4.—Average passenger car and motor truck license fees paid in 28 States, first six months of 1924

The gasoline tax receipts for 1923 were $36,813,939, of which amount the State highway departments directed the expenditure of $21,528,559, or only 58 per cent. In 13 States the State highway departments control the expenditure of all the gasoline tax receipts, in 5 States they control from 90 to 99 per cent, in 6 States 50 to 89 per cent, and in 8 they control less than 50 per cent. In 3 States, Alabama, North Dakota,

TABLE 8.—*Gross motor vehicle registration receipts and amounts applicable to highway work by or under the supervision of State highway departments, 1923*

State	Gross registration receipts	Applicable to highway work by or under the supervision of State highway departments.	
		Amount	Percentage of receipts
			Per cent
Alabama	$1,541,018	$1,204,449	78
Arizona	281,671	281,671	100
Arkansas	1,435,090	430,527	30
California	10,608,544	4,906,015	46
Colorado	1,126,219	534,954	47
Connecticut	4,329,432	4,329,432	100
Delaware	516,209	516,209	100
Florida	1,963,066	1,394,529	71
Georgia	2,156,406	2,095,763	97
Idaho	914,015	229,840	25
Illinois	9,653,796	9,653,796	100
Indiana	3,693,715	3,492,498	95
Iowa	8,827,063	8,000,000	90
Kansas	3,435,606	1,750,000	51
Kentucky	2,678,733	2,678,733	100
Louisiana	2,191,241	2,191,241	100
Maine	1,660,268	1,474,383	89
Maryland	3,536,955	3,183,260	90
Massachusetts	6,989,633	6,639,155	95
Michigan	10,500,786	4,741,625	45
Minnesota	7,316,772	7,316,772	100
Mississippi	1,077,616	580,852	54
Missouri	4,016,384	4,016,384	100
Montana	729,621	73,326	10
Nebraska	3,353,175	2,932,243	87
Nevada	153,888	144,992	94
New Hampshire	1,571,327	1,464,097	93
New Jersey	7,653,780	7,515,116	98
New Mexico	295,000	280,250	95
New York	19,862,442	14,896,831	75
North Carolina	3,728,045	3,700,000	99
North Dakota	760,852	760,444	100
Ohio	9,662,370	4,832,962	50
Oklahoma	3,217,771	2,895,000	90
Oregon	4,069,609	2,924,707	72
Pennsylvania	15,844,304	15,844,304	100
Rhode Island	1,286,659	1,196,909	93
South Carolina	902,609	722,087	80
South Dakota	1,130,959	1,055,176	93
Tennessee	2,049,653	2,028,806	99
Texas	5,441,509	2,368,569	43
Utah	430,105	430,105	100
Vermont	938,860	860,803	92
Virginia	3,200,162	3,200,162	100
Washington	3,898,598	3,741,168	96
West Virginia	2,608,508	2,608,508	100
Wisconsin	4,958,934	4,693,887	95
Wyoming	414,096	414,096	100
	188,613,074	153,226,636	81

and Pennsylvania the State highway departments do not receive any part of the gasoline tax receipts, nor do they have any supervision over their expenditure.

The revenues derived from license fees and gasoline taxes which are not turned over to State highway departments are usually divided among the counties or are credited to the general fund, from which they are again appropriated for highway work or for the payment of interest and the retirement of highway bonds. In only a few cases are the motor-vehicle revenues diverted to uses wholly foreign to road work.

RELATION OF HIGHWAY EXPENDITURES TO INCOME

In discussing the subject of highway expenditures the question is often raised as to the amount of money which the people of a State or a community can afford to spend for the improvement of the roads. The relationship which exists between the highway expenditures and the income of the people reduced to a per capita basis furnishes one criterion which can be applied.

In Table 10 are set forth by States the per capita highway expenditures exclusive of interest and principal payments, and the percentage ratios which these highway expenditures bear to the per capita incomes of the several States. In 13 States—Alabama, California, Connecticut, Illinois, Maryland, Massachusetts, Missouri, New Hampshire, New Jersey, New York, Pennsylvania, Rhode Island, Vermont—the per capita highway expenditures in 1921 amounted to less than 1 per cent of each person's income; in 17 States—Colorado, Florida, Georgia, Kentucky, Louisiana, Maine, Michigan, Nebraska, New Mexico, Ohio, Oklahoma, Tennessee, Texas, Utah, Virginia, Washington, West Virginia—the highway expenditures were between 1 and 2 per cent of the income; in 11 States—Delaware, Indiana, Kansas, Minnesota, Montana, Nevada, North Carolina, North Dakota, South Carolina, Wisconsin, Wyoming—the highway expenditures were between 2 and 3 per cent; and in 7 States—Arizona, Arkansas, Idaho, Iowa, Mississippi, Oregon, South Dakota—they were over 3 per cent. South Dakota, where the highway expenditures amounted to 6.84 per cent of the people's income, showed the highest ratio; in New York, where the ratio is lowest, the highway expenditures were only 0.34 per cent of the income. In general, it may be said that where the per capita highway expenditures are large, there the ratio between such expenditures and the per capita income is also high. In States where there are large cities the per capita highway expenditures are low in general, while the per capita income is greater than in the State where the rural population dominates. In the New England and Middle Atlantic States the per capita expenditures for highway work averaged 0.55 per cent of the per capita income; the average for all

TABLE 9.—*Gross gasoline-tax receipts and amounts applicable to highway work by or under the supervision of State highway departments, 1923*

State	Gross gasoline-tax receipts	Applicable to highway work by or under the supervision of State highway departments	
		Amount	Percentage of receipts
			Per cent
Alabama	$1,133,085		
Arizona	474,123	$118,531	25
Arkansas	1,219,199	301,095	25
California	2,518,893	1,259,446	50
Colorado	846,353	402,018	47
Connecticut	880,223	880,223	100
Delaware	88,579	88,579	100
Florida	1,641,042	1,150,356	70
Georgia	1,502,503	247,666	16
Idaho	396,487	396,487	100
Indiana	2,906,428	2,514,756	86
Kentucky	680,435	680,435	100
Louisiana	754,438	754,438	100
Maine	286,077	285,840	100
Maryland	688,304	688,304	100
Mississippi	467,855	187,142	40
Montana	441,249	75,877	17
Nevada	115,843	60,000	52
New Hampshire	163,065	161,823	99
New Mexico	165,000	156,750	95
North Carolina	2,909,905	2,900,000	100
North Dakota	461,082		
Oklahoma	599,000	599,000	100
Oregon	1,958,141	1,885,421	96
Pennsylvania	5,491,523		
South Carolina	1,511,453	411,328	27
South Dakota	624,692	565,000	90
Tennessee	812,357	801,502	99
Texas	1,215,623	911,717	75
Utah	404,086	106,903	26
Vermont	168,173	168,173	100
Virginia	1,556,921	1,037,947	67
Washington	1,225,150	1,225,150	100
West Virginia	366,490	366,490	100
Wyoming	140,162	140,162	100
	36,813,939	21,528,559	58

other sections of the country was over 1 per cent; and in the West North Central and the Mountain States it was over 2 per cent. These higher percentages represent the effort that is being made by small populations to bear the costs of extended highway improvement with relatively small income per capita.

TABLE 10.—*Per capita highway expenditures and ratio of expenditure to income, 1921*

State	Highway expenditures per capita	Ratio of per capita highway expenditures to per capita income
		Per cent
Alabama	$2.10	0.61
Arizona	29.30	3.86
Arkansas	11.10	3.52
California	10.70	.74
Colorado	9.50	1.08
Connecticut	6.10	.59
Delaware	25.00	2.76
Florida	8.80	1.55
Georgia	5.00	1.58
Idaho	25.00	4.03
Illinois	5.80	.57
Indiana	15.10	2.56
Iowa	16.40	3.71
Kansas	12.60	2.15
Kentucky	4.80	1.05
Louisiana	6.60	1.32
Maine	10.70	1.34
Maryland	6.20	.66
Massachusetts	4.80	.40
Michigan	13.80	1.69
Minnesota	15.60	2.33
Mississippi	9.60	3.34
Missouri	4.50	.71
Montana	16.90	2.85
Nebraska	8.00	1.43
Nevada	25.50	2.71
New Hampshire	11.70	.92
New Jersey	8.40	.77
New Mexico	9.40	1.79
New York	4.90	.34
North Carolina	10.00	2.61
North Dakota	11.20	2.65
Ohio	11.30	1.40
Oklahoma	6.90	1.44
Oregon	33.80	3.90
Pennsylvania	7.90	.84
Rhode Island	4.50	.37
South Carolina	5.60	2.34
South Dakota	21.70	6.84
Tennessee	5.20	1.25
Texas	9.80	1.66
Utah	10.20	1.55
Vermont	6.10	.82
Virginia	6.20	1.32
Washington	16.40	1.68
West Virginia	6.30	1.03
Wisconsin	15.50	2.17
Wyoming	22.30	2.23
Average	9.00	1.13

RECOMMENDED INVESTIGATIONS IN THE FIELD OF MOTOR VEHICLE TAXES

The data available at this time relative to the whole subject of motor vehicle taxation are confined solely to the revenues obtained through license and registration fees and the gasoline tax. In a great many States the motor vehicle owner has to pay additional taxes on his car. In the first place, he is subject to a personal property tax on his car the same as any other kind of property, except in States where the legislature has provided specific exemption for the automobile from this kind of tax. So far as can be learned, there are now 13 States which do not levy a property tax on the motor vehicle—Delaware, Idaho, Iowa, Michigan, Minnesota, New Hampshire, New York, North Dakota, Oklahoma, Oregon, Pennsylvania, Tennessee, and Vermont. In these States the license fees are collected in lieu of property taxes. No definite figures have as yet been ascertained as to the amount of the general property taxes on automobiles for the country as a whole. The estimates which have been made show wide variations. It is frequently stated that even those States which tax the motor vehicle as property fail to a very considerable extent in collecting the taxes from all of the car owners. In many instances there does not appear to be any close relationship between the State motor vehicle registration office and the taxing authorities. To remedy this condition was the reason given in a number of the States for increasing license fees and exempting the automobile from the general property tax. To be able to show fully the extent to which the motorists contribute towards highway expenditures, it is necessary to make further inquiries into this subject of the taxation of the motor vehicle as property.

In the second place, motorists in many cities are obliged to pay special municipal license fees, sometimes known as "wheelage taxes." In some cases they amount to more than the State license fees. It is therefore of importance to collect data as to the amount of revenues derived from the automobile in this manner and the use to which such funds are put.

Further investigations should also be made with reference to the distribution of license fees and gasoline taxes as between State highway authorities and local governmental units. Municipalities, in some instances, already are claiming a share of the gasoline taxes. In order to work out some fundamental principles regarding the allocation of such revenues, it is necessary to determine more fully than has yet been done the relative use which city-owned motor vehicles make of our rural highways as compared with rural cars, and data should be collected further which will show the proportion of the total motor vehicle mileage which occurs within city limits as compared with the rural highway mileage. Such facts will be of use in the development of apportionment principles.

The effect which improved and hard-surfaced roads have on values of adjacent farm lands constitutes a problem which is of special interest to one studying this subject of the equitable apportionment of the highway finance burden. There are now under way several researches inaugurated by the Bureau of Public Roads in cooperation with State bodies, which it is expected will provide certain fundamental data which will be of value in determining how much real property in the rural sections should contribute towards highway improvement.

There is no doubt that many of the State legislatures will find it necessary in the next few years to revise and expand their highway laws and to provide more adequately for contemplated highway improvements. In order that such legislative work may be done according to sound economic principles and in harmony with good public policy, it behooves those who are interested in these subjects to procure and interpret all the underlying facts of highway finance so that they may be available to all who seek guidance and knowledge therefrom.

HIGHWAY TRANSPORTATION COURSES THEIR PLACE AND CONTENT

By CHARLES LEE RAPER, Franklin Professor of Transportation, and Dean of the College of Business Administration, Syracuse University

I BELIEVE it is correct to ask a college to offer a course in any subject which has large importance in the life of a community. If this be a correct basis for a college course, transportation should have at least one course in many of our colleges. To transport people and their products is equally as important as the production of their goods or the development of their surroundings. The movement of people and their products is an essential function of civilized life. A college should, in my judgment, offer courses dealing with the essential functions of life.

Many colleges give some attention, in a general way, to the value of transportation. Some of them give specific attention to transportation by rail or by water. Upon the basis of relative importance, highway transportion deserves a place in the college courses. The traffic which moves over the highways is at least large enough to entitle it to be compared with the traffic which moves over the railways or the waterways. People are transporting themselves more and more over their highways by horse vehicle or motor car, and I am convinced that this process will continue to grow. They are sending their commodities in ever greater quantity over the highways. The highway is, and always will be, a very important form of transportation. To construct more of this form of transportation and to keep it ever in service—this constitutes one of the big tasks of our day, and we have no doubt that our children's children will think of it as one of their great tasks.

The coming of rapid transportation, on rail or water, has been one of the revolutionary things in modern life. It has made new things; it has remade old things. The building of efficient highways for use by animal-drawn vehicles, and especially for motor-driven vehicles, may well prove itself a thing of revolutionary force.

HIGHWAY TRANSPORTATION BOTH AN ENGINEERING AND A BUSINESS PROPOSITION

The construction, maintenance, and use of the highway constitute both an engineering and a business proposition. The construction and maintenance are primarily questions of engineering. The use of the highway is essentially a business question. The construction and maintenance of the highway involve much cost, and the financing of this cost is a business problem, whether this be done by a corporation or by the State. The operation of vehicles upon the highway is primarily a matter of business for the individual or group of individuals. Highway transportation courses should, therefore, cover both the engineering aspects and the business aspects.

The location and construction of the highways should be given in the engineering group of courses rather than in the business administration group. The cost of the roads and their effect upon the movement of people and goods should, I think, be considered to some degree in the engineering courses.

The use of the highway and the traffic which moves upon it should be considered by the business administration group of courses. The vehicle and its performance are essentially business problems, even though these affect the location, construction, and maintenance of the highway. Both the highway and the vehicle must be considered, I believe, to some degree in both groups of courses—the engineering group and the business administration group.

The location, construction, and maintenance of the highway are essentially things of public interest. The use of the highway is, on the other hand, more largely a matter of private interest. How much of the cost of construction and maintenance should be borne by the users of the road and how much by the taxpayers should be adequately considered in any course in highway transportation. I think this is particularly true with the courses offered by the business administration group.

The question as to whether the vehicle which uses the highway should be defined in law as a public carrier and should, therefore, be held responsible for its services as a public carrier, even though its owner is an individual or corporate citizen, is also a fundamental one and must be considered in any course in highway transportation.

OPERATION OF HIGHWAY TRANSPORTATION

The establishment and operation of a vehicle service on the highway should be considered more particularly as a private interest. For this reason any course offered in the business administration group should give careful and somewhat detailed consideration to at least the following aspects:

The vehicle.—The kind of vehicle, its size, weight, or tonnage capacity, constitute an important problem or series of problems for the operator of the vehicle and for the public. All of these aspects have large influence upon the owner's profits and also upon the cost of the construction and maintenance of the highway, as well as upon the safety of those who use the highway.

The routing of the vehicle.—The routing of the vehicle is primarily a business question, and it is of large importance. Upon the solution of this question depends, to a large degree, the prosperity of the operator of the vehicle. It is necessary that the routing of a vehicle be decided upon after a careful survey of the traffic from the point of view of the present and the future. It should also be decided upon after a study of the roads, actual and potential, and the amount of competition. The problem of the routing of a vehicle, as well as that of its size, weight, and tonnage capacity, should have adequate consideration in any course in highway transportation offered in a college of business administration.

The cost system for operation and maintenance of the vehicle.—Not only is it necessary for the operator of the vehicle to give his best attention to the vehicle and to its routing, but it is also necessary that he give his best attention to his cost system. Any course in highway transportation given in a college of business administration should, I think, devote considerable attention to a study of the cost systems for highway vehicle operators.

Rates for the service.—Rates for highway transportation services are as fundamentally important as rates for any other carrier or as prices for any other business. It should be clear to the users of the highway vehicles as well as to their operators that the rates should be large enough to cover all the items of cost of operation and to provide something for profits. Any course in highway transportation which may be offered by a college of business administration should go into a study of rates with no little emphasis. The following factors in rate making should at least have proper consideration:

1. Adequate interest on the investment in the highway vehicle.
2. Adequate depreciation of the vehicle and its auxilliary equipment.
3. Adequate provision for the maintenance of the vehicle.
4. Proper provision for taxes and license fees, which should be paid to the State for the use of the highway.
5. The cost of operation, including insurance on vehicle and traffic, supervision, etc.

The competition of highway transportation.—The rates for the highway service and the type of the service should always be considered in connection with the competition of this service with that of other carriers—as, for example, railways. This should be done for the sake of the business interest of the highway operator and for the sake of the public. The public now carefully regulates railway service. It should also regulate, in a large measure, the highway service. Both the highway service and the railway service should work in the largest possible cooperation. Sharp competition between these two types of transportation is injurious to both carriers and the public.

The contents of a highway transportation course, as they have been enumerated, might seem to suggest a very considerable value toward graduation. I believe that they can be adequately treated in a three-hour course for one-half of a college year. This is, of course, upon the assumption that the student has had effective training in the more general aspects of economics and politics.

NOTE.—Contributed to the round-table conference on highway economics at the meeting of the American Economic Association, Chicago, Ill., December 30, 1924.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets or to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage in the United States, 1914. A Summary.
*393. Economic Surveys of County Highway Improvement. 35c.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*555. Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials. 10c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
74. State Highway Mileage and Expenditures for the Calendar Year 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D–2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D–4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D–6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D–13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D–15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

DECEMBER 31, 1924

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.53	$ 2,186,247.54	464.1	$ 766,721.98	$ 380,609.23	85.3	$ 15,433,392.54	$ 7,486,096.43	849.7	$ 79,416.15	$ 39,708.07	0.4	$ 1,150,401.73	Alabama
Arizona	8,338,355.41	4,287,683.88	527.8	713,173.87	406,825.70	37.5	1,757,774.35	1,064,861.24	150.4	900,984.42	550,592.13	70.5	1,185,738.05	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	1,637,568.79	688,383.74	68.1	6,463,537.33	2,551,817.83	339.5	922,438.41	429,429.67	74.6	868,423.13	Arkansas
California	12,999,075.03	5,547,148.17	533.7	4,463,144.73	2,347,870.47	177.3	11,194,859.15	5,768,442.68	359.2	491,988.13	287,811.77	1.9	3,042,032.91	California
Colorado	8,108,070.31	4,029,898.97	502.6	880,626.15	444,393.39	47.6	6,398,940.51	2,937,913.88	212.0	52,776.78	35,229.34	5.8	2,112,445.42	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	199,024.56	98,423.00	4.9	3,262,054.48	1,045,804.04	54.1				967,409.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	462,969.09	197,825.82	13.8	1,315,585.49	504,231.10	33.1				29,758.26	Delaware
Florida	961,134.07	461,470.92	48.8	1,416,010.74	681,331.64	25.4	8,274,647.58	4,060,548.44	236.4	398,642.71	171,123.94	10.3	912,412.06	Florida
Georgia	17,167,373.32	7,955,805.20	1,214.2	1,587,490.88	755,094.39	138.3	10,735,267.33	5,121,998.59	753.6	820,931.13	379,916.19	39.0	227,082.63	Georgia
Idaho	8,151,697.92	4,092,395.62	506.8	243,857.57	139,962.62	18.7	2,549,392.79	1,530,324.99	171.9	449,282.31	292,975.38	29.9	622,052.49	Idaho
Illinois	26,954,705.06	12,279,546.33	804.7	3,588,051.46	1,751,750.91	109.8	13,539,290.92	5,724,129.81	469.4	53,053.82	26,531.91	2.0	2,544,533.04	Illinois
Indiana	7,577,444.16	3,656,540.97	225.7	2,427,983.85	1,155,095.08	83.6	14,873,492.00	7,295,074.86	475.3				2,206,681.09	Indiana
Iowa	23,195,778.19	9,237,031.86	1,582.9	1,986,999.59	899,447.75	157.7	7,146,594.05	3,264,649.47	501.9	2,098,815.48	900,900.00	78.9	1,034,107.92	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	5,234,527.81	2,014,461.76	159.4	14,772,222.55	6,939,501.97	523.5	2,808,795.68	1,295,068.71	151.5	7,079.76	Kansas
Kentucky	10,822,980.31	4,513,947.28	429.4	1,992,917.60	801,849.55	71.8	8,167,464.30	3,744,306.32	313.4	373,194.00	153,936.89	13.0	1,057,698.86	Kentucky
Louisiana	8,488,453.18	3,636,143.36	561.2	1,368,277.16	662,014.97	127.1	4,808,154.95	2,389,108.72	284.0	309,676.07	154,838.03	1.5	423,336.92	Louisiana
Maine	6,911,058.78	3,299,936.38	230.7	719,052.70	343,786.54	30.3	1,298,297.12	622,267.36	44.6				823,982.72	Maine
Maryland	5,750,044.42	3,213,321.78	243.2	358,411.02	179,205.49	15.4	2,801,989.31	1,199,249.89	85.3	171,075.41	52,750.00	4.2	4,422.84	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	318,961.44	111,675.42	5.2	6,457,832.90	2,053,941.50	105.7	475,794.54	151,122.03	9.2	1,487,313.83	Massachusetts
Michigan	13,434,136.07	6,060,612.23	494.6	715,646.16	349,245.07	29.1	15,805,464.85	7,475,927.56	534.6				1,992,986.14	Michigan
Minnesota	24,037,561.24	9,885,843.07	2,292.0	6,121,718.17	2,724,798.97	421.3	5,354,043.16	2,605,800.00	540.7	904,876.83	99,800.00	100.1	2,176.96	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	1,456,853.97	694,789.48	99.1	8,175,190.45	4,081,320.47	481.7	740,381.06	370,190.51	60.3	656,031.15	Mississippi
Missouri	11,352,027.70	6,245,899.18	803.5	1,326,969.31	654,897.27	100.6	23,384,211.22	10,198,390.71	878.0	3,849,064.62	1,104,000.14	141.8	737,000.70	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	911,797.03	637,863.31	91.3	2,245,492.02	1,561,934.43	200.5	1,038,200.22	590,707.56	117.2	3,691,576.48	Montana
Nebraska	7,876,337.16	3,714,591.69	1,440.4	382,998.91	178,915.76	27.0	7,880,402.23	3,857,680.61	831.6	916,341.27	458,170.60	109.1	3,241,487.45	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	1,102,421.45	928,932.16	92.2	4,584,901.22	3,867,550.75	399.1	38,940.11	32,787.34	2.0	207,325.77	Nevada
New Hampshire	3,075,750.19	1,487,867.58	171.3	767,035.04	352,053.17	27.3	1,015,014.26	478,844.00	31.6	29,611.26	13,680.00	0.9	102,519.26	New Hampshire
New Jersey	7,623,795.12	2,651,531.49	148.7	1,613,508.11	445,405.00	25.8	9,024,358.54	2,567,236.24	64.0				815,074.27	New Jersey
New Mexico	5,305,286.45	2,758,849.68	714.3	912,328.07	559,290.38	84.7	6,556,272.99	4,386,862.23	566.1	250,983.39	154,354.77	16.7	729,974.94	New Mexico
New York	18,852,742.49	8,257,844.44	572.7	1,675,708.28	673,352.21	48.3	30,276,154.84	11,395,122.88	573.4	5,534,338.00	1,567,205.00	101.5	4,814,623.47	New York
North Carolina	12,667,732.97	5,676,757.55	884.7	3,582,873.36	1,217,143.39	95.9	8,717,956.58	3,503,481.05	244.5	1,437,527.05	669,797.82	52.1	1,221,071.08	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1,587.9	1,298,143.59	631,116.50	264.8	2,994,555.33	1,502,937.30	407.8	295,074.10	147,537.01	27.2	1,663,559.77	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	3,530,795.03	1,357,034.24	96.1	12,576,796.19	4,693,490.80	357.1	1,402,203.38	529,000.00	40.5	1,680,720.97	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	2,082,033.66	1,016,086.84	92.1	8,041,755.84	3,838,019.43	340.7	2,638,022.45	1,029,576.18	145.9	764,058.52	Oklahoma
Oregon	12,082,873.17	5,819,093.79	555.6	1,209,085.39	707,463.39	95.4	2,835,165.45	1,563,370.68	140.9	239,530.50	110,954.29	9.3	205,276.85	Oregon
Pennsylvania	36,825,248.98	14,114,594.79	729.7	1,575,929.93	740,322.44	38.5	22,786,424.84	5,088,062.50	374.5	3,143,714.81	887,575.00	58.8	2,770,361.27	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	744,122.20	309,710.13	15.7	1,295,720.34	371,919.84	17.9				472,184.07	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	1,588,812.61	730,417.82	236.3	5,447,225.87	2,224,762.61	351.4	963,573.52	175,677.28	91.9	432,543.07	South Carolina
South Dakota	8,674,597.85	4,244,536.27	989.8	1,957,845.88	1,042,131.02	298.2	5,942,471.71	3,349,284.11	929.3	244,890.99	16,103.39	46.1	66,525.21	South Dakota
Tennessee	5,805,583.35	3,313,936.07	259.6	4,660,828.80	2,300,390.95	161.0	11,845,838.75	5,354,964.13	404.1	1,141,855.70	570,929.32	75.3	474,415.53	Tennessee
Texas	42,341,998.66	15,190,524.91	3,122.8	6,220,993.23	2,044,321.25	347.0	23,948,858.75	9,700,550.27	1,457.1	5,100,820.57	2,173,554.55	332.0	1,515,162.02	Texas
Utah	3,304,423.75	1,895,805.92	219.0	1,172,195.60	722,070.17	124.1	4,261,594.27	2,606,774.61	245.2	322,811.39	221,330.96	27.1	470,491.34	Utah
Vermont	1,922,114.16	942,769.12	74.4	152,908.38	75,454.18	6.3	2,104,586.81	996,894.18	51.4	53,364.64	25,682.31	1.4	491,179.21	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	1,265,937.26	592,715.20	55.1	9,515,581.51	4,465,484.83	326.1	1,119,227.82	508,734.98	38.6	224,235.56	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	927,405.97	405,402.07	46.3	3,433,300.97	1,615,300.00	130.9	265,662.59	132,600.00	12.4	442,480.48	Washington
West Virginia	5,489,747.95	2,365,041.63	255.6	1,555,196.35	715,520.21	55.5	4,473,244.28	1,945,014.09	143.1	77,099.55	27,448.00	0.4	700,008.17	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1,325.3	1,324,722.32	628,581.41	51.7	4,007,973.67	1,933,333.37	156.3	135,408.28	67,703.00	7.2	3,607,799.66	Wisconsin
Wyoming	5,127,625.51	3,078,098.70	687.6	1,631,342.21	935,575.29	148.2	3,891,144.34	2,423,252.08	264.6				250,424.93	Wyoming
Hawaii													365,625.00	Hawaii
TOTALS	$549,555,391.27	$237,852,399.82	32,452.9	$82,832,049.26	$ 38,453,007.84	4,664.3	$ 394,971,329.14	$ 176,574,933.88	17837.4	$ 42,360,339.14	$ 16,618,135.17	2,108.6	$ 55,626,527.29	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 109,511,199.84 Federal aid $ 48,628,269.57 Miles 4,550.6

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 5, NO. 12 ▽ FEBRUARY, 1925

"OH, TO BE IN ENGLAND NOW THAT APRIL'S THERE, —"

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1925

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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IMPRESSIONS OF ENGLISH HIGHWAY PRACTICE

By A. B. FLETCHER, Consulting Highway Engineer, United States Bureau of Public Roads

This article by Mr. Fletcher, the first of a series, to be published in Public Roads, dealing with his observations of English highway practice, was prepared as a paper to be read at the Twenty-second Annual Meeting of the American Road Builders' Association. In it he presents in a pleasant and interesting way some of the more vivid of his impressions gained in a visit to England in the spring of 1924. In subsequent articles he will deal in greater detail with several of the subjects to which he here refers.

DURING THE spring of 1924 I was detailed to study the rural roads of England. With this not unpleasant assignment I was employed during the months of April, May, and June, and although England did not live up to her reputation for fine spring weather—it rained nearly every day—it was possible to get about without particular difficulty even in the remote country districts. One could forgive the rain for the astonishing beauty of the roadsides which it produced.

In addition to meeting the English road officials, the trip included a journey in a Chandler (American-made) automobile from London to Edinburgh, and a little farther north in Scotland, going up through the counties on the east side of the Little Island and returning to London by way of the westside counties, a drive of more than 1,500 miles.

The main north and south roads were rather generally followed, but the large centers of population and the manufacturing cities were avoided for two reasons. I wanted particularly to see the rural roads; and the heavy traffic congesting the narrow, crooked streets of the cities, built when riding in a saddle was more popular than any other sort of transportation, was not tempting to an amateur driver whose forbears for some generations had been taught to drive on the right-hand side of the street.

They say that even Henry Ford had to yield to British conservatism and put the steering wheel on the right-hand side before the English would buy his cars. The Chandler car, originally made with the steering wheel on the left side, had been remodeled so that the wheel, clutch pedal, and brake were moved to the right side. The gear shift, however, was left in the center and had to be worked with the left hand. One soon learned to make the car go, but the idiosyncrasies of the car, together with passing other cars on the wrong side of the road, made such a thing as intuitive driving out of the question. It was not difficult, however, to go as fast as the law permits, for in England as in Massachusetts the legal limit of speed is 20 miles an hour. The law is obeyed equally well in both places, I should say.

The journey, as it was planned, gave an opportunity to inspect a considerable mileage of the two main north and south trunk lines throughout the length of England.

There are many similarities and some differences between the English country roads and the rural roads of the United States. They are perhaps more crooked even than the roads of our older States, and the reason for the poor alignment is apparently the same in both countries. In neither country was there any thought of motor traffic or any other sort of fast traffic when

The narrow, crooked streets of the medieval cities, built when riding in a saddle was popular, are not tempting to an amateur driver

the roads were surfaced. Mostly the roads were improved by putting down hard surfaces within the limits of the then existing rights of way. We seem to be making faster progress in this country in correcting that fault, perhaps not because we are more progressive, as we like to think, but because the urge is greater. The motors have come upon us at a faster rate and in greater numbers, relatively, than in England.

Great Britain still has from 20 to 25 per cent of its highway freight moved by horses, while I suppose that in the United States not more than 10 per cent of the traffic is horse-drawn, and in some of the States the horses are no longer counted in the traffic census.

But when it comes to the matter of the riding quality of the roads we have very much to learn from England. I saw no road on my long auto journey so rough as are most of our rural roads, but it should also be said that I saw hardly any so smooth as the best of the roads in the United States built by the State highway departments with the Federal-aid stimulant.

THE ENGLISH AND AMERICAN ROAD PROBLEMS COMPARED

Our road problem is so much bigger than Great Britain's that the reason for the better average improvement of the English road is apparent. In all England, Wales, and Scotland, there are but 177,000 miles of road, cities and boroughs included, as against our estimated mileage of 2,941,000 outside of the cities and towns. England, Wales, and Scotland have an average of about 242 persons to the mile of road, while in this country there are not more than 35 people to the mile. In Massachusetts, one of our States of dense population, there are about 175 people to the mile.

The English roadside almost invariably is a thing of beauty, and an American has to go to Scotland before he feels at home. For some reason, sparse population and lack of money, perhaps, the Scotch roadsides are nearly or quite as barren and unkempt as

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On the whole we mark our roads better than the English. The rectangular object at the right is a dust-covered mirror

most of ours are. The English roads generally have a wide grass border, and there are trees and shrubs everywhere. Sometimes the line of sight is restricted by the roadside growths, but it is plain to understand why even then the shrubs are spared.

The drainage water from the roads disappears quickly from the carriageway and flows off in unseen ditches near the right-of-way lines. The turf at the pavement edge is carefully trimmed and kept so just as in a park. Laborers trimming the edge with spades with a tightly stretched cord for a guide were seen, working as painstakingly as if they were trimming a garden border.

The traffic control at bad road intersections in the country as handled by the agents of the automobile clubs, in cooperation with the police authorities, is wonderfully well done and worthy of much more attention than can be given to it here. In the matter of road signs, however, I was disappointed. I think that on the whole we mark our roads, at least so far as direction signs are concerned, better than it is done in England.

Nearly all of the roads inspected were of some bituminous type, tar-mac, tar macadam, asphalt, tar-painted, etc. In my 1,500-mile journey not more than a mile or two of the road in the open country was recognizable as being of the cement-concrete type, and some of that had been covered with tar or asphalt. I do not imply that no cement-concrete surfaces have been laid on the rural roads, but seemingly most of the work of that type must be in the cities and towns. The English road officials from Sir Henry Mayberry, chief of the road department of the Ministry of Transport, down to the surveyors of the smaller counties seem to be almost a unit in believing that they can not afford to scrap the great mileage of bituminized roads which they have constructed even if it can be proved that the concrete type is more desirable from the viewpoint of maintenance costs, which they seem to disbelieve. They seem to be thoroughly wedded to the bituminous types of construction.

The reason for their preference is clear when one sees the carefully planned grades, long established, with sodded shoulders, drainage ditches and entering driveways, and when one realizes the great expense which they have incurred in putting in the heavy road foundations. Notwithstanding the large costs of maintaining the bituminous road surfaces, the Englishman is slow to adopt a road type with which he is not familiar, and he is entirely willing that his American cousin shall make what he calls the experiments. He wants proof of the reputed low cost of maintenance of concrete roads, and in his doubting conservatism he will not admit that the relatively few years of life of the American concrete roads have given them any "history" worth talking much about.

THE COST OF MAINTAINING ENGLISH BITUMINOUS ROADS

How large the upkeep charges are for the bituminous roads is shown in some figures published by the ministry showing the annual cost of upkeep on four class 1 roads leading out of London into the Provinces.[1] The total length of the four roads is 321 miles and the report says:

> The annual cost of upkeep taken over the whole length of each road ranges from £700 per mile in one case to £980 in another ($3,360 to $4,704 per mile).

Assuming the average width of the carriageway of these roads to be 30 feet, and assuming that $4,000 per mile fairly represents the average annual cost of upkeep, we see that these four roads cost not less than 22½ cents per square yard for maintenance. This annual outlay would appear to be sufficient to renew completely the wearing surface as often as once in five years. The figures, the report says, do not include any capital outlay for the roads in the past.

I have been unable to find a statement showing the mileage or square yardage of concrete roads in Great Britain. The handbook of the British Portland Cement Association states that 281 concrete roads had been built up to June, 1923. Of these, nearly 79 per cent were built after the year 1920. Many of the roads were very short, some less than one-half mile in length, and evidently the total yardage was not sufficiently impressive to be set forth in the handbook.

On the other hand, much of the new arterial road work in the vicinity of London is of the cement-concrete type, some of it 50 feet in width with the slab 8 inches thick, and the work very well executed, some American equipment being employed.

There are so many types of bituminous road in use or offered for use in England that the patentees have had to tax their ingenuity to find names for them all, but from my observation I believe that the great bulk of the pavements are either some sort of penetration macadam, tar-mac, or merely surface-painted.

I was much interested in looking for wavy conditions or corrugations in the surface of the bituminous roads. The county surveyors will tell one that they have waves in their pavements, and they seem to know what one is talking about when one speaks of corrugations, but their waves are not like our waves, for I found very little, practically no, evidence of the corrugations which are so prevalent in our bituminous pavements.

Their methods of spreading the bituminous material are much like ours except that they work more slowly than we do, perhaps more carefully and skillfully, but

[1] Ministry of Transport Report on the administration of the road fund for the year 1922-23.

I am forced to the conclusion that the apparent superiority of the British bituminous roads is due very largely to their thick, heavy foundations and in some measure to the use of curbs to confine the pavements at the sides.

Almost without exception the English road is built with what they call "hard core" as a foundation. Hard core may consist of almost any hard material laid as a foundation for the full width of the carriageway. The stones are large, sometimes as large as 8 inches in longest dimension and often as large as the thickness of the layer will permit. The hard core layer is usually from 8 to 12 inches in thickness. Strong hard slag seems to be a popular material, but when that is too costly and brickbats or stones from walls or buildings are available they are put into the road. The point is, of course, to secure a hard unyielding base which will not hold capillary water. The county surveyors are beginning to wonder if these hard core foundations, strong as from our viewpoint they seem to be, are going to be heavy enough for the future motor traffic. When we consider how few of our rural roads have any foundation at all under the 5 or 6 inches of bituminized stones, do we need to look much further for the cause of the corrugations? Or to explain the apparent superiority of the British roads?

The English seem also to be completely convinced of the need of substantial curbs to prevent the lateral movement of the pavement. All of the new work with which the Ministry of Transport has to do is provided with curbs, and the county surveyors generally are installing curbs in connection with their widening work and extensive repairs.

BASES 12 INCHES THICK

Illustrating the extreme care in the matter of road foundations which some of the county surveyors are taking, C. F. Gettings, of Worcester County, told me when I was looking over some of his work with him that because of the bad subsoil with which he has to deal, he first lays a stratum of "blinder," or cinders, 3 inches thick over the subgrade, followed by a layer of slag 6 inches thick, then a layer of 3-inch slag to a thickness of 3 inches, then 3 inches of tar-mac, and finally a dusting on the top of pulverized slag. Thus he has 12 inches of material in place before he lays the wearing course, which he prefers shall be tar-mac. Tar-mac is crushed slag, heated and mixed with a refined tar at the works where the slag is produced and shipped cold to the highway job.

When conditions permit him to do so, Mr. Gettings employs what we in this country have come to call the stage-construction method. First, after rolling as much as is effective the 6-inch layer of slag, he turns on the traffic to further consolidate it. He does the same with the 3-inch slag layer, sometimes giving it a light tar treatment and allowing the traffic to pass over it for a considerable period, but not after it shows any sign of distress. In this manner he makes sure, before the wearing course is laid, that he has a firm, hard base for it, and that there will probably be no further settlement of his foundation after the pavement is completed.

Some of the best bituminous pavements that I have seen anywhere were built under Mr. Getting's direction. The traffic over the main Worcester County roads is called heavy. The country is in the Midlands,

English roads have a wide grass border, and there are trees and shrubs everywhere. A pile of hard core material has here been left by the roadside

one of the regions of great manufacturing activity. The traffic census taken by the ministry in August, 1923, indicates that the roads are in the group of 1,000 to 1,500 tons per 16-hour day. We would not consider that to be a very heavy traffic, but Mr. Gettings thinks a carriageway 22 feet wide and of the thickness before stated is needed for the main roads of the country where the subsoil is bad.

THE COST OF ENGLISH LABOR

In the United States such substantial work would cost much more than the public is accustomed to pay for the rural highways. The English feel the high cost of their road work, too. Common labor in 1924 was receiving the equivalent of 25 cents per hour, a price which the English employer thought was outrageous, yet we at that time were paying more than double the English hourly wage. Living standards and cost of living are different, but I do not believe the disparity is so great as I had been led to believe.

In July, 1924, Portland cement cost in London about $2.22 per barrel, American basis, and other materials of construction seemed to be not greatly lower in price than in the United States.

The arterial roads near London are of great interest. They are being built in part to supply the general need for more roads, in part to by-pass through traffic so that it will not have to go through the narrow, already congested streets of the metropolis, and in part to provide work for the unemployed.

In England, in 1924, there were more than 1,000,000 persons "on the dole," or supported to a greater or less extent by the Government. Any public work which could be found for these unemployed was welcomed, and for several years the construction of the arterial roads in the Greater London area and the by-pass roads around the cities and towns in the Provinces has provided work for many men. In 1922–23 there was set aside more than $31,000,000 for the road-fund unemployment program.

In the Greater London area alone, 165 miles of the arterial roads, including the widening and straightening of some roads, are either under construction or planned for, the total estimated cost of the work being in the vicinity of $60,000,000.

All of this work is being done on a large scale. Rights of way 100 to 120 feet in width are being secured and with much delay and difficulty. When houses are in the way and must be demolished, the public authorities must provide other houses elsewhere to shelter the tenants, so great is the housing shortage.

The carriageways of the most important arterial roads are to be 50 feet wide and curbed. Sidewalks and planting strips are provided for, and iron fences are installed along the right-of-way lines. On the Great West Road all pipes, sewers, water and gas and all electric wires are to be placed in conduits under the sidewalks and planting strips. One section of this road under traffic in 1924 was said to have cost at the rate of £180,000 ($864,000) per mile.

In the arterial road and by-pass work very low grades are insisted upon, the alignment is as nearly perfect as can be obtained, and no effort seems to be spared in securing the best results in all branches of the work. The pavements, or many of them, are of the cement-concrete-base type laid in most instances with the expectancy of covering them later on with asphalt, but in some cases the concrete is being allowed to take the traffic for the present. The concrete slab, 8 inches thick and reinforced, is said to be costing about 10s. per square yard ($2.40 approximately).

ENGLISH ROAD ADMINISTRATION

The Ministry of Transport took the place of the road board in 1919, and under Parliament it is the highest road authority of Great Britain. Its organization is somewhat like that of the Bureau of Public Roads. The road department of the ministry is in charge of a chief, Sir Henry Mayberry, with Col. C. H. Bressey under him in the capacity of chief engineer who, in turn, has a corps of divisional engineers located at various places throughout the country in direct charge of the operations.

The revenue which the department has for road purposes, derived almost wholly from the registration fees paid on account of the motor vehicles, amounted in 1924 to about £15,000,000 ($72,000,000). This is about the same sum that Congress has been appropriating recently for our Federal-aid work, but here the likeness ends. Colonel Bressey told me that the annual revenue which the department received represented, fairly closely, one-third of the total sum spent annually by Great Britain for all highway purposes. Such a sum, approximately $216,000,000, would not go very far toward paying the annual highway bill of the United States, which in 1922 was estimated to amount to more than $1,000,000,000.

The ministry has divided the roads into two categories, known as class 1 and class 2, and the present policy is to allot to the counties not more than 50 per cent of the cost of improvements on class 1 roads, and not more than 25 per cent to such work on the class 2 roads. Roads less important than class 2 roads are merely local in character, and they receive no money from the ministry.

In England, Wales, and Scotland, the total mileage of class 1 roads is about 23,000 miles, and the class 2 roads aggregate about 14,000 miles. The total mileage of all roads, including the merely local ones, is given as 177,321 miles, so, roughly speaking, the ministry is concerned with about 21 per cent of the total mileage of the country.

GASOLINE TAX ABANDONED IN 1921

Prior to January 1, 1921, at which time the present road fund was established, there had been in effect a tax on gasoline or "motor spirit," speaking in the language of the country, by means of which most of the grants made by the ministry were financed. After the year 1915 this tax was at the rate of 6 d., about 12 cents per gallon. Beginning with January 1, 1921, the tax was abolished, and in place of the gasoline tax as a revenue producer a tax of £1 (about $4.80) per horsepower of the motor vehicles was substituted.

This tax is still in effect, and the owner of a Ford car, for example, pays into the public treasury annually very nearly $100 for the privilege of driving on the British roads. The high registration fee has fostered the manufacture and use of low-powered cars, and special attention has been given to small-cylindered motors and high piston speeds.

The ministry does not favor a proposed plan to return to a gasoline tax, which the motor interests are pressing for, chiefly, I believe, because the officials dislike to abandon a source of assured income for a plan which they think to be less sure. They say they need at least £15,000,000 per annum for the roads; that the present taxing plan will surely produce that revenue; and that their experience with the collection of the gasoline tax prior to 1921 has not left happy memories. The old relatively high gasoline tax was doubtless evaded in many instances. Sir Henry Mayberry says that while the motors were increasing in numbers from year to year in an astonishing fashion the receipts from the tax remained nearly constant. Much of the gasoline and the kerosene imported into England nominally for heating and manufacturing purposes doubtless found its way into the tanks of the motor cars.

To conclude this somewhat sketchy and superficial summary of some rather large subjects, I believe that in speed of road construction, in the matter of road equipment of all kinds, as concerns motor-vehicle regulation, highway financing, and research and experimental work generally, we do not have much to learn from Great Britain.

In matters of road location we can see there in aggravated form the same sort of mistakes which have been made in this country, particularly in the older States, where we have put down expensive pavements on faulty locations with unnecessarily tortuous alignment, a timid following of the line of least resistance, using rights of way good enough, perhaps, when horses did the work but sadly inadequate for our present-day motorized traffic.

It is doubtful if we can hope to equal the bituminous roads of England until we pay more attention to the foundations. We should either follow somewhat after the English methods or develop some substitute, possibly less costly, which will be as effective.

PERCENTAGE OF WATER FREEZABLE IN SOILS

By A. M. WINTERMYER, United States Bureau of Public Roads

RECENT experiments by the United States Bureau of Public Roads to determine the percentage of water that will freeze at ordinary freezing temperatures in various typical soils have shed new light on probable relations between certain distinguishable characteristics of the soils and the percentage of their contained water that can be frozen.

When a soil "freezes," i. e., when water contained in the soil is frozen, the freezing very seldom involves all the water in the soil. Most soils contain some percentage of water, which may be large or small, depending upon the nature of the soil, that will not freeze at temperatures immediately below the freezing point of water; and in some soils a portion of the contained moisture can not be frozen even at temperatures below −78° C.

If the soil be frozen in a dilatometer, using the method employed in the recent Bureau of Public Roads experiments, which will be later described, the water content can be divided into three volumes determined by the temperature at which they are frozen. The first volume, which will be frozen at 0° C., is classified as free water; the second, which will freeze at from −4° C. to −78° C., is classified as capillary or adsorbed water; and the third, classified as combined water, is so intimately associated with the soil that it can not be frozen even at temperatures below −78° C. Different soils differ widely in the percentages of their contained water which fall into these three classes. In some the entire water content can be frozen at 0° C. or slightly below. Clean standard Ottawa sand is such a soil. In other materials, especially those high in soluble salts, with which the contained water combines, the percentage of such salts may be so large that no part of the water may be frozen even at very low temperatures.

As the freezing of water in the soils composing road subgrades, with the accompanying phenomenon of heaving or swelling, is one of the most troublesome of highway problems, it is decidedly worth while to ascertain what properties of the soil affect the percentage of the contained water that may be frozen, for by doing so it is possible that means may be found to alter these properties in such a way as greatly to reduce the troublesome freezing. Yet very little work has been done along this line; indeed, the only published information that has been discovered is that contained in Technical Bulletin No. 36 of the Michigan Agricultural College, by George J. Bouyoucos.

PERCENTAGE OF WATER FROZEN CLOSELY RELATED TO DYE ADSORPTION VALUE OF SOIL

The principal results of the recent experiments by the Bureau of Public Roads are the indication of a fairly close relation between the percentage of water frozen at −1.5° C. and the dye adsorption number of the soil, and the perfecting of the dilatometer method by which the percentage of water frozen may be determined with reasonable accuracy.

It appears as a general rule from these tests that the percentage of water in soil freezable at −1.5° C. increases as the dye adsorption number of the soil decreases. Apparently the adsorption of dye and the water frozen are both controlled by the same properties of the soil, namely, the chemical composition, colloidal content, mineralogical composition, organic-matter content, percentage of soluble and insoluble salts, etc. The presence of certain constituents or properties which give the soil a high adsorption number causes also the removal of a certain amount of water from the active state, so that the freezing point is lowered very considerably, allowing only a small percentage to be frozen at −1.5° C. In a number of instances in the recent experiments the percentage of soluble salt was so large that no moisture was frozen in the soil even after lowering the temperature to −20° C.

There seems to be a tendency also for the percentage of water frozen to be greater in coarse-grained than in fine-grained soils; but to this tendency there are many exceptions. The mechanical analysis may show a large percentage of coarse material, yet only a small amount of moisture may be frozen. The mechanical analysis shows only the arbitrary size of the soil particles, and not their real condition as, for instance, whether the particles are crystalline or colloidal, etc. A much closer relationship seems to exist between the adsorption number and the moisture frozen; and this is not surprising when we consider that the same factors control both the dye adsorbed and the moisture frozen.

The experiments show also that moisture does not freeze with the same speed in different soils. In some soils the freezing is very rapid; in others very slow. The real freezing point is not at 0° C., but somewhat lower; in some cases it was found to be below −1.5° C.

Since the dilatometer method, as perfected in these experiments, accurately shows the amount of free or active water in the soil, it would seem advisable to determine this amount of water in the soil in conjunction with the regular soil analysis. It is this water that gives rise to the troubles so often encountered during the freezing season. After the active water has been determined in a particular soil and found to be high in amount, methods may possibly be devised to remove the greater part of this water from the active state by adding some suitable material that will form a combination with it, and lower the freezing point considerably. This would greatly decrease the amount of water frozen and minimize the danger of upheavals.

THE RESULTS OF THE TESTS

In an earlier series of tests, conducted in 1922, the various soil samples were treated with the same amount of water, 5 cubic centimeters in all cases. It was found, however, that this quantity would not moisten the entire sample in all cases. It was decided therefore at the commencement of these tests to try the use of several percentages of water with respect to the volume of the soil, corresponding to the moisture equivalent, and the percentages obtained in the vertical capillarity and water-holding capacity tests,[1] and to run a sufficient number of tests to determine which would be the most favorable percentage for use. On the basis of these trial tests, the percentage of moisture corresponding to the indication of the vertical capillarity test was finally chosen for use in the subsequent tests.

The results of the tests on the various soil samples treated in this way are shown in Table 1, which gives, in addition to the quantity of moisture added to the soil and the percentage frozen, the mechanical analysis and the absorption number of the soil.

[1] For descriptions of these tests, see PUBLIC ROADS, vol. 4, No. 3, July, 1921.

TABLE 1.—*Relation between mechanical analyses and adsorption numbers and water frozen in various soils*

Soil No.	Total sand	Total fine material	Adsorption number	Water added	Water frozen	Water frozen
	Per cent	*Per cent*		*Cubic centimeters*	*Cubic centimeters*	*Per cent*
3	38.8	61.2	15.0	6.5	3.45	53.0
4	52.0	48.0	35.0	6.0	3.60	60.0
5	70.4	29.6	7.5	4.0	2.90	72.5
6	41.2	58.8	22.5	6.5	4.70	72.3
7	12.8	87.2	40.0	7.5	4.00	53.4
8	2.8	97.2	30.0	7.5	4.05	54.0
9	4.4	95.6	22.5	7.0	4.50	64.3
10	15.6	84.4	22.5	7.0	4.20	60.0
11	31.2	68.8	8.3	5.5	3.30	60.0
12	37.6	62.4	17.5	7.0	4.50	64.3
13	12.0	88.0	17.5	8.0	6.15	77.0
14	26.8	73.2	55.0	8.0	3.80	47.5
15	26.4	73.6	22.5	7.0	4.30	61.5
16	34.0	66.0	80.0	7.5	4.00	53.4
18	88.1	11.9	11.0	7.0	5.40	77.2
19	28.4	71.6	15.0	5.5	2.45	44.5
20	9.2	90.8	130.0	7.0	1.70	24.3
22	18.0	82.0	90.0	7.5	3.60	48.0
23	47.2	52.8	57.5	8.5	5.35	63.0
24	22.4	77.6	90.0	9.0	5.20	57.8
25	28.4	71.6	150.0	10.5	3.20	30.5
26	15.2	84.8	158.8	9.0	3.60	40.0
27	20.8	79.2	127.5	8.5	3.74	44.0
28	22.0	78.0	45.0	6.5	4.00	61.5
29	66.0	34.0	25.0	6.0	4.24	70.7
31	13.2	86.8	100.0	8.5	3.10	36.5
32	53.6	46.4	42.5	6.0	2.85	47.5
33	3.6	96.4	90.0	9.0	5.30	58.9
36	41.6	58.4	31.3	8.0	3.70	46.3
37	23.2	76.8	55.0	8.0	4.00	50.0
38	90.4	9.6	7.5	5.0	2.55	51.0
40	40.0	60.0	30.0	7.5	3.60	48.0
41	46.8	53.2	16.3	9.0	3.20	35.6
42	8.4	91.6	120.0	11.7	5.30	45.3
43	8.8	91.2	132.5	8.0	2.00	25.0
44	18.0	82.0	140.0	8.0	2.65	33.1
50	17.6	82.4	38.8	9.0	5.00	55.5
51	38.0	62.0	52.5	7.0	4.00	57.2
54	.9	99.1	52.5	8.0	4.40	55.0
55	14.4	85.6	55.0	7.0	4.00	57.2
56	6.0	94.0	18.8	8.5	5.90	69.4
59	8.8	91.2	40.0	7.5	5.20	69.3
60	12.0	88.0	87.5	8.5	5.40	63.5
67	16.0	84.0	130.0	9.0	4.80	53.4
70	2.6	97.4	55.0	8.0	2.10	26.3
80	31.6	68.4	22.5	6.5	4.05	62.3
92	55.2	44.8	35.0	8.0	4.55	57.0
101	7.2	92.8	87.5	11.0	6.60	60.0
102	4.5	95.5	95.0	8.5	4.70	55.3
103	9.0	91.0	85.0	9.0	6.00	66.7
119	14.4	95.6	20.0	8.0	5.35	66.9
120	1.6	98.4	50.0	10.0	5.75	57.5
138				6.5	2.60	40.0
139				6.0	3.40	56.7
157	35.6	64.4	110.0	8.0	4.20	52.5
158	94.0	6.0	2.5	4.5	3.40	75.6
186	43.6	56.4	35.0	8.0	4.30	53.8
188	78.4	21.6	15.0	5.0	3.75	75.0
200	26.8	73.2	57.5	8.0	5.60	70.0
201	70.8	29.2	15.0	6.0	4.20	70.0
202	70.0	30.0	25.0	3.5	2.35	67.2
290	74.4	25.6	6.3	5.0	4.20	84.0
291	65.6	34.4	7.5	5.5	3.25	59.1
292	30.0	70.0	32.5	13.0	6.20	47.7
293	74.8	25.2	7.5	5.0	4.05	81.0
294	77.6	22.4	30.0	5.0	3.45	69.0
323	30.6	69.4	11.3	9.0	4.00	44.5
324	16.5	83.5	17.5	10.5	5.30	50.5
325	49.4	50.6	11.3	8.0	5.00	62.5
326	29.9	70.1	12.5	8.0	4.10	51.3
327	52.8	47.2	12.5	8.0	4.20	52.5
328	54.1	45.9	10.0	9.0	5.90	65.5
329	24.2	75.8	13.8	9.0	4.90	54.5
410	15.8	84.2	32.5	8.0	0	[1] 0
411	3.1	96.9	57.5	10.0	0	[1] 0
412	6.4	93.6	25.0	9.0	0	[1] 0
413	2.7	97.3	40.0	10.0	0	[1] 0
414	7.5	92.5	43.8	9.0	0	[1] 0
415	3.3	96.7	50.0	11.0	0	[1] 0
416	3.4	96.6	68.8	10.0	0	[1] 0
417	15.1	84.9	38.7	10.0	0	[1] 0
418	2.9	97.1	67.5	9.0	0	[1] 0
419	23.7	76.3	35.0	7.0	0	[1] 0
420	38.8	61.2	27.5	8.0	0	[1] 0
422	10.6	89.4	31.3	9.0	0	[1] 0
423	2.5	97.5	32.5	12.0	0	[1] 0
424	5.6	94.4	32.5	10.0	0	[1] 0
425	5.3	94.7	58.7	8.0	0	[1] 0
426	3.0	97.0	55.0	10.0	0	[1] 0
427	3.7	96.3	50.0	9.0	0	[1] 0
428	1.4	98.6	67.5	12.0	0	[1] 0
430	10.8	89.2	90.0	9.0	0	[1] 0
431	7.2	92.8	43.8	10.0	0	[1] 0
486	72.0	28.0	10.0	6.5	3.00	46.2
489	59.6	40.4	9.0	5.0	2.90	58.0
492	81.2	18.8	3.0	3.5	2.20	62.9
496	64.6	35.4	9.0	5.5	3.90	70.9
499	71.7	28.3	4.0	5.0	3.60	72.0
500	69.9	30.1	7.0	5.0	2.70	54.0
502	68.1	31.9	9.0	5.5	2.40	43.6
504	71.2	28.8	3.0	5.0	3.70	74.0
507	53.4	46.6	10.0	5.5	3.20	58.1
508	52.0	48.0	11.0	6.0	3.60	60.0
510	84.2	15.8	2.0	4.0	3.00	75.0
518	88.7	11.3	2.0	4.0	3.20	80.0
523	64.9	35.1	3.0	4.0	2.40	60.0
524	63.2	36.8	6.5	4.5	2.30	51.1
527	63.8	36.2	9.0	6.0	4.50	75.0
530	63.6	36.4	6.0	4.5	2.90	64.5
535	55.2	44.8	11.0	6.0	3.00	50.0
536	53.2	46.8	5.5	4.0	2.30	57.5
538	48.4	51.6	4.0	4.5	2.90	64.5
540	72.5	27.5	2.0	3.5	2.00	57.2
541	63.2	36.8	4.0	4.5	3.40	75.6
542	68.9	31.1	6.0	4.0	2.80	70.0
556	43.0	57.0	10.0	6.0	3.40	56.7
559	43.6	56.4	13.0	6.0	3.10	51.7
562	35.4	64.6	22.0	8.0	4.30	53.8
565	51.9	48.1	16.0	6.5	3.70	56.9
568	44.8	55.2	13.0	6.0	2.40	40.0
569	52.9	47.1	14.0	8.0	4.00	50.0
570	56.2	43.8	12.0	6.0	4.50	75.0
571	74.2	25.8	8.0	4.5	3.90	86.7
572	65.0	35.0	6.0	5.0	3.60	72.0
583	64.6	35.4	27.0	7.0	3.50	50.0
588	71.0	29.0	8.0	4.0	3.20	80.0
591	76.6	23.4	7.0	4.0	2.80	70.0
592	68.2	31.8	16.0	5.0	3.00	60.0
593	64.8	35.2	11.5	4.0	1.60	40.0
595	56.7	43.3	8.0	4.5	3.00	66.7
740	64.6	35.4	7.0	9.0	6.30	70.0
741	75.7	24.3	11.7	9.5	3.80	40.0
742	62.7	37.3	11.7	11.5	3.70	32.2
743	52.8	47.2	19.0	9.0	4.00	44.5
747	61.4	38.6	4.0	8.0	5.60	70.0
749	71.5	28.5	4.5	9.0	6.90	76.7
752	15.9	84.1	23.8	15.0	6.70	44.7
753	49.7	50.3	11.2	4.0	.80	20.0
756	2.9	97.1	55.0	11.0	0	[1] 0
757	3.4	96.6	68.7	11.0	0	[1] 0
758	6.9	93.1	32.5	10.5	0	[1] 0
759	6.6	93.4	40.0	9.0	0	[1] 0
760	2.2	97.8	50.0	10.0	0	[2] 0
761	3.3	96.7	35.0	10.0	0	[2] 0
762	2.1	97.9	40.0	11.5	0	[1] 0
763	3.8	96.2	31.3	9.5	0	[1] 0
766	10.7	89.3	27.5	8.0	3.60	45.0
772	19.4	80.6	42.5	8.0	4.10	51.3
Quartz sand	100.0					100.0

TABLE 1.—*Relation between mechanical analyses and absorption numbers and water frozen in various soils*—Continued

[1] These soils were all from the Great Salt Desert in Utah. They contain varying percentages of salt, all so high that all water mixed with the samples undoubtedly combined with the salt, so that none of it could be frozen at −1.5° C.

[2] These soils were from Nevada and were similar to the Utah soils.

Considering the data in Table 1, it will be seen that the amount of water frozen varied from 100 to 0 per cent. The quartz sand, however, was the only soil in which 100 per cent was frozen, and upon grinding this standard Ottawa sand into flour only 70 per cent of the added water was frozen. This seems to indicate that the size of the particles is one of the controlling factors in the amount of water frozen; and, as a general rule, it will be observed that the percentage of water frozen does increase from the fine to the coarse-textured soil. But while there is such a general tendency there is apparently no definite relation between the mechanical analysis and the percentage of water frozen. For example, soils Nos. 5, 201, 290, and 293 all have a high percentage of coarse material and show a high percentage of water frozen. Soil No. 38 is practically all coarse material, yet less of the contained moisture was frozen than in some of the soils which contained only a small amount of coarse material, as, for example, soils Nos. 9, 10, and 59. Soil No. 20 is composed of a large percentage of fine material, and only a small amount of the water was frozen, thus conforming more closely to the general rule than the majority of the soils.

The fact is, of course, that many factors must be considered as influencing the percentage of water that can be frozen at −1.5° C., since soil is more often

heterogeneous than homogeneous. As a rule the condition of the moisture and the temperature at which it will be frozen depend upon the physical, chemical, and mineralogical composition of the soil. Soluble salts, organic matter, and colloidal material present will remove a certain percentage of the moisture from the free state and prevent its freezing.

MECHANICAL ANALYSIS DOES NOT INDICATE FREEZING PERCENTAGE

The mechanical analysis merely shows the size of the soil particles, while the water unfrozen is an index not only of the size of the particles but of other properties as well. Since the adsorption number of the soil indicates in the main the same properties, it might be expected that a relation would exist between the adsorption number and the percentage of water frozen; and, in fact, such a relation is evident in the results of the tests, as will be observed by reference to Table 2, in which certain of the data from Table 1 are arranged in the descending order of the percentage of water frozen.

TABLE 2.—*Comparison of percentage of moisture frozen and adsorption numbers of soils*

Soil No.	Moisture frozen		Adsorption number		Clay content	
	Each sample	Average	Each sample	Average	Each sample	Average
	Per cent	*Per cent*			*Per cent*	*Per cent*
290	84.0	82.5	6.3	6.9	16.0	15.8
293	81.0		7.5		14.4	
13	77.0	72.9	17.5	20.3	46.0	25.1
158	75.6		2.5		5.2	
188	75.0		15.0		15.9	
5	72.5		7.5		11.6	
6	72.3		22.5		32.0	
29	70.7		25.0		29.3	
200	70.0		57.5		40.4	
201	70.0		15.0		20.4	
56	69.4	65.1	18.8	31.8	28.0	35.9
59	69.3		40.0		34.8	
294	69.0		30.0		13.6	
202	67.2		25.0		18.0	
119	66.9		20.0		45.6	
12	64.3		17.5		36.8	
60	63.5		87.5		71.8	
23	63.0		57.5		40.8	
80	62.3		22.5		41.8	
15	61.5		22.5		41.6	
11	60.0		8.3		22.4	
291	59.1	55.8	7.5	60.4	19.2	42.5
24	57.8		90.0		52.8	
120	57.5		50.0		76.5	
55	57.2		55.0		36.0	
92	57.0		35.0		32.0	
50	55.5		38.8		38.8	
54	55.0		52.5		33.8	
186	53.8		35.0		35.0	
67	53.4		130.0		55.6	
157	52.5		110.0		45.6	
22	48.0	45.6	90.0	74.7	56.8	51.4
292	47.7		32.5		63.1	
32	47.5		42.5		33.2	
14	47.5		55.0		48.8	
36	46.3		31.3		36.4	
42	45.3		120.0		82.0	
19	44.5		15.0		29.6	
27	44.0		127.5		52.0	
26	40.0		158.8		60.4	
44	33.1	31.8	140.0	145.0	71.6	58.7
25	30.5		150.0		45.7	

Even in the data for the individual soil samples in this table there is evident a general relation. Comparing the average values for the several groups, this relation becomes more apparent, for these values show very definitely that as the adsorption number and the clay content of the soil increase the percentage of moisture frozen decreases. Several of the individual values depart from the rule, but no more than one would expect in dealing with a relation in which there are involved so many factors. In this connection the fact that certain classes of dyes are adsorbed more readily by some kinds of soil than by others must also be taken into consideration. The chemical composition of the soil has a great deal to do with the adsorption. Acid dyes, as a rule, are more strongly adsorbed by basic minerals and soils than are basic dyes, and vice versa. In cases where ammonia exists in the soil as a salt the adsorption may be increased in some instances; in others decreased by the presence of the salt. In these tests methyl violet, a basic dye, was used; and it is quite probable that some of the departures from the general rule in individual samples may be attributed to the basic properties of the dye. Certainly, one would expect that the basic soils would show a relatively lower adsorption number than the acid soils.

TEMPERATURE WHICH SOILS MAY ATTAIN WITHOUT BEING FROZEN

It is known that pure water may be easily cooled to −3° or −4° C. without the appearance of ice if kept quite still while the temperature is reduced. While conducting the freezing experiments it was decided to determine the temperature to which the soils could be lowered and yet remain unfrozen.

The procedure was the same as in the other tests except that the temperature was steadily lowered instead of being maintained at −1.5° C., and the soil was left undisturbed in the freezing mixture. The results obtained showed that all the soils thus treated, about 20 in number, remained unfrozen until the temperature reached −4° C., when they automatically froze. The soil, however, may be kept in the surfused or undercooled condition an indefinite length of time, and the process of solidification can be started by the slightest vibration or movement. This shows that the soil may very often remain unfrozen even though the temperature be below the freezing point.

EFFECT OF HEATING ON THE AMOUNT OF WATER FROZEN

With a view to determining the effect of heating on the amount of water freezable in the soil samples of a red clay soil were heated to temperatures of 100, 200, 300, 400, 500, and 600° C., respectively, and maintained at these temperatures for one hour, after which water was added in amounts corresponding to the vertical capillarity percentage, and the wet samples were frozen. The heating changed the color of the soil from a light red at 100° C. to a dark brick red at 600° C., and the plastic properties of the soil decreased with increase of temperature.

At 100° and 200° C. the soil was very plastic and froze after considerable agitation at −1.6° and −1.5° C., respectively. The time required for completion of the freezing process after it commenced was in each case 50 minutes.

At 300° C. there was a slight decrease in the plasticity of the soil and the color changed to a slightly darker red. Freezing commenced at −1.5° C. only after considerable agitation, and the time required to complete the process was 60 minutes.

At 400° C. the soil was less plastic and turned to a dark brick red. It froze readily at −1.5° C., requiring 50 minutes.

At 500° C. the plastic qualities were destroyed, the color remaining about the same as that heated to 400° C. The soil froze readily at −1.5° C., and the time required was 1 hour and 30 minutes.

At 600° C. the soil was nonplastic, the color becoming a shade darker. It froze readily at −1.5° C. and required about one and three-quarters hours.

APPARATUS AND METHODS EMPLOYED IN THE TESTS

The apparatus employed in the experiments consisted as shown in Figure 1, of a bath, A, dilatometer, D, and potentiometer, C. The type of dilatometer used had a removable stem with ground joint, as shown in Figure 2. This facilitated the handling and cleaning of the apparatus and removed one of the chief causes of breakage. The dial of the potentiometer used to record the soil temperature was so graduated that the potential difference could be read in

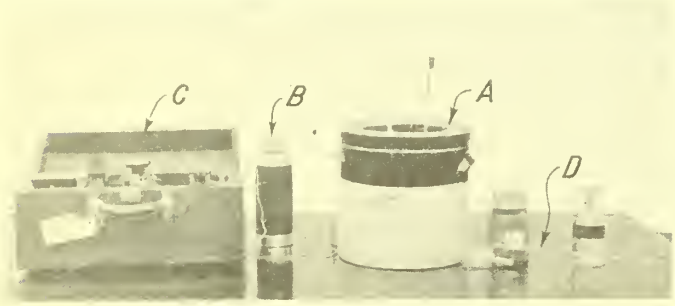

FIG. 1.—Apparatus employed in the dilatometer test

tenths of microvolts. With this arrangement the temperature was controlled to within one-tenth of a degree. For controlling the cold junction temperature an ordinary thermos flask (fig. 1, B) was provided.

The temperature bath consisted of two earthenware jars (fig. 1, A) placed one inside the other. The outer jar was of 3 gallons capacity and the inner of 1 gallon capacity. The space between the two jars was packed tightly with hair, the upper part being made waterproof by a layer of paraffin. The cooling mixture consisted of crushed ice and salt.

In the preliminary tests conducted in 1922 great difficulty was experienced in obtaining concordant results. The variation in some soils was so great that the data obtained were of no value. This variation was found to be due chiefly to the nonuniformity of the sample. Another factor found greatly to aid in obtaining close checks was to let the sample stand overnight and thus insure a more equal distribution of the water added. This method was followed during the final work.

The method of quartering as used is as follows: An aliquot sample of air-dried soil, about 500 grams, was mixed as thoroughly as possible and then quartered. Two opposite quarters were then mixed together thoroughly and again quartered. This procedure was continued until about 25 grams of the original sample remained. This was then thoroughly mixed and the required 25 grams weighed. By following this procedure and by allowing the wet soil to stand overnight very close, and in some cases, perfect checks were obtained. As already stated, the amount of water added in the recorded tests corresponded to the vertical capillarity percentage.

The main procedure was as follows: To 25 grams of air-dry soil the required amount of water was added. The soil and water was thoroughly mixed in the dilatometer. The thermocouple and stopper were then inserted into the neck of the bulb and sealed and allowed to stand overnight. Ligroin or petroleum naphtha was then added through the stem, so that the bulb was just completely filled. The dilatometer was then mildly jarred to set free any air bubbles held by the soil. Finally gentle suction was applied, which quite completely exhausted the air from the soil. The air was excluded from the bulb by filling it and letting the air escape at the bend. When no more air could be expelled, the stem was filled with ligroin and covered with a paraffined cork cap to prevent volatilization. The ligroin was used to fill the bulb and act as an indicator, because it is not miscible with water.

The bulb of the dilatometer was then placed in the freezing bath and the soil allowed to supercool to $-1.5°$ C. When this temperature was reached, the water in the soil was caused to freeze by gently moving the dilatometer in the freezing mixture until solidification commenced, which was indicated by the rise of the ligroin in the stem. The bulb was allowed to remain in the bath, with frequent shaking, until the rise of the ligroin ceased. The total rise represented the total amount of expansion due to the formation of ice. This water frozen at $-1.5°$ C. is regarded as free water.

In order to determine the factor for converting the volume of expansion due to the formation of ice, 5 cubic centimeters of distilled water was added to 25 grams of clean, dry Ottawa sand. Upon freezing this gave an expansion of 0.5 cubic centimeter, showing that 1 cubic centimeter of water will expand 0.1 cubic centimeter upon freezing. This value corresponded to that obtained in the experimental stage of the work.

The time required to complete a determination varied with the class of soil. The speed of solidification at $-1.5°$ C. is very low and the greatest amount of time consumed is after crystallization commences. It varies from one-half hour to two hours.

In order to obtain concordant results and eliminate trouble, certain precautions must be observed. The sample should always be thoroughly mixed so that the quantity used will represent a uniform mixture of the whole. As nearly as possible the soil should be allowed to attain the same supercooling temperature. There is little danger of premature solidification at $-1.5°$ C.

The ice bath should be kept dry and constant, at least within 1° below the required temperature. This can be accomplished by siphoning off the water collecting at the bottom of the bath.

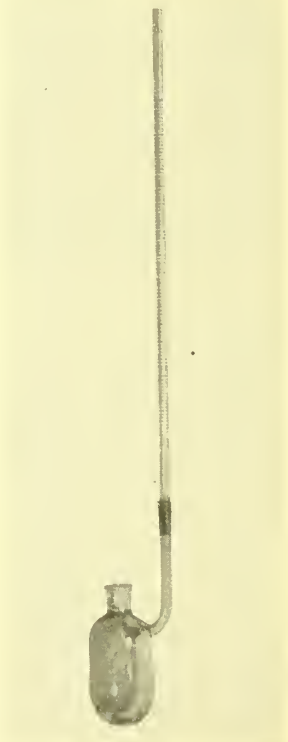

FIG. 2.—Modified dilatometer

Most of the trouble experienced is due to the cork stoppers. These do not always form a tight connection with the neck of the bulb, because of internal unsoundness of the cork or small cracks on the side. It is always best to use sound corks, because the ligroin will be forced through these internal or external openings by the pressure within the bulb and will loosen the paraffin seal. Should this happen, it is better to use a new cork than to try to renew the seal, because it is seldom accomplished. The sides of the corks should also be examined for paraffin. If any is found, it should be removed, because it will be dissolved by the ligroin and allow it to reach the seal and escape.

A STUDY OF MOTOR-VEHICLE ACCIDENTS IN MONTANA, OREGON, AND WASHINGTON

By A. C. ROSE, Associate Highway Engineer, United States Bureau of Public Roads

STATISTICS of highway accidents involving motor vehicles which seem to point clearly to congestion of traffic as the principal cause have recently been compiled by the writer from newspaper reports of accidents in the States of Montana, Oregon, and Washington during the greater part of the period from December, 1923, to September, 1924. A classification of the reports, procured from State-wide newspaper clipping services, indicates the relative importance of other causes, such as speed, recklessness, and carelessness

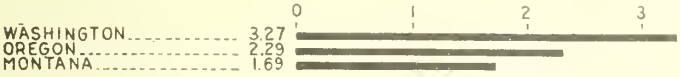

FIG. 1.—Number of accidents per 1,000 motor vehicles

or incompetence of drivers, intoxication, mechanical breakdowns, faulty highway conditions, etc., but these seem to be secondary causes which augment the fundamental condition.

The most striking fact revealed by the compilation is that the accident rate per 1,000 motor vehicles is lowest in Montana, in which the number of motors vehicles registered is lowest, and is higher in the other two States in almost direct proportion to the number of vehicles registered in these States. This indication seems to be directly contrary to the annual statistics of motor-vehicle accidents collected nationally, which show a reduction in the rate of accidents as the registration has increased year by year. The opposing indications of the two compilations are clearly brought out in the following tabulation:

ANNUAL STATISTICS

Year	Number of deaths	Motor-vehicle registration	Fatalities per 1,000 vehicles
1918	7,525	6,146,617	1.23
1919	7,968	7,565,446	1.05
1920	9,103	9,231,941	.99
1921	10,168	10,463,295	.97
1922	11,666	12,238,375	.95
1923	14,412	15,092,177	.96

LOCAL STATISTICS

State	Number of accidents	Motor-vehicle registration	Accidents per 1,000 vehicles
Montana	135	69,100	1.95
Oregon	459	161,739	2.84
Washington	1,012	265,541	3.82
Total	1,606	496,380	3.23

It is possible that the explanation of the difference is that the annual statistics express the salutary results of better traffic regulation, growing familiarity with the motor vehicle, increased caution, and improvements in the design of vehicles and highways; while the State or local statistics, being for the same time and for approximately equal states of advancement with respect to the above factors, express the natural result of differences in traffic density. The probability that this explanation is correct seems to be strengthened by an examination of the particular causes of the accidents

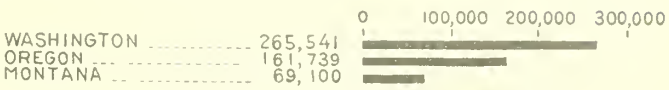

FIG. 2.—Motor-vehicle registration, July 1, 1924

as classified in Table 6, from which it appears that the principal cause contributing to the increase in the accident rate is the element of recklessness and carelessness, a human fault nor eradicated by regulation, and not greatly affected by increase of knowledge, but one which becomes more and more potent for disaster as the traffic density increases.

CROWDED ROADS INCREASE ACCIDENT RISK

Considering the summary of the State statistics for the months of December, 1923, and January, April, May, June, July, August, and September, 1924, which are given in Table 1, it is to be remembered that the States represented are among the sparsely settled areas of the West. Yet even in this section it will be observed the accident rate is 3.23 per 1,000 vehicles over an eight-month period. Conservatively estimated for a year the rate becomes 4.5 accidents per 1,000 vehicles. In other words, the average driver in these States has 1 chance in 222 every year to meet with an accident. Assuming the span of driving years over an average lifetime to be 30, the conclusion is that in the Pacific Northwest one driver in every seven is liable to accident during a lifetime of driving. And for the driver who travels the more congested highways the risk is considerably greater.

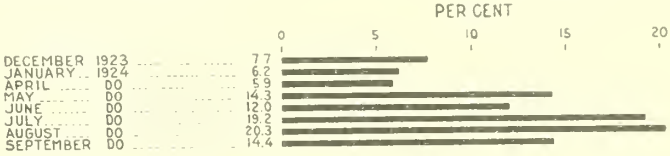

FIG. 3.—Total number of accidents in Montana, Oregon, and Washington, by months

TABLE 1.—*General summary of motor-vehicle accident data*

	State			Total
	Montana	Oregon	Washington	
Number of accidents	135	459	1,012	1,606
Motor-vehicle registration [1]	69,100	161,739	265,541	496,380
Number of accidents per 1,000 vehicles	1.95	2.84	3.81	3.23
Number of fatalities	33	56	104	193
Ratio of fatalities to accidents (per cent)	24.4	12.2	10.3	12.0
Number of fatalities per 10,000 motor vehicles	4.78	3.46	3.92	3.89
Number of persons injured	168	448	1,035	1,651
Ratio of persons injured to accidents (per cent)	124.4	97.6	102.2	102.8
Number of persons injured per 10,000 motor vehicles	24.3	27.7	39.0	33.2
Number of vehicles damaged	103	386	932	1,421
Ratio of vehicles damaged to accidents (per cent)	76.2	84.1	92.1	88.4
Vehicles damaged per 10,000 motor vehicles	14.9	23.9	35.1	28.6
Mileage of roads	64,732	45,475	45,816	156,023
Motor vehicles per mile of roads	1.1	3.6	5.8	3.2
Population, 1920 census	548,889	783,389	1,356,621	2,688,899
Persons per motor vehicle	7.9	4.8	5.1	5.4

[1] Registration for first six months of 1924 as reported in PUBLIC ROADS, vol. 5, No. 7, September, 1924.

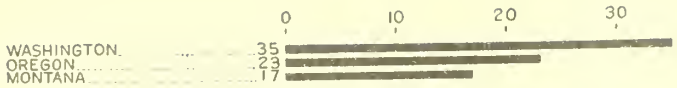

FIG. 4.—Number of cities in three States with population over 2,500

The influence of congestion is suggested by the summarized statistics and by Figures 1 and 2, which show that the frequency of accidents becomes increasingly greater as the number of motor vehicles increases. The suggestion is strengthened by Figure 3, which shows that the greatest number of accidents occurs in the months of July, August, and September, when the roads bear the heaviest traffic, although a comparison of these data with traffic estimates over the entire year seems to indicate that the number of

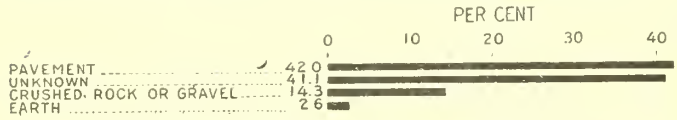

FIG. 5.—Percentage of the total number of accidents in the three States on various types of road

accidents per 1,000 motor cars is greater during the winter months. The previous conclusion may also be inferred from Figure 4, in which it is shown that the number of accidents per 1,000 vehicles increases with the number of cities over 2,500 population. And this inference is again confirmed by the data presented in Figure 5, which show that the greatest number of accidents occurs on paved roads, which, radiating from the more populous centers or serving as the main trans-state traffic lanes, carry the densest traffic.

TIME OF ACCIDENTS

A number of studies have been made to determine whether the accident risk is greater during the day or the night. Table 2 is a summary of the findings over the eight months' period. While these figures are interesting, the study has not been sufficiently detailed to be conclusive.

TABLE 2.—*Effect of time of day on causation of accidents*

State	Motor-vehicle accidents						Total	
	Daylight		Darkness		Unknown			
	Number	Per cent	Number	Per cent	Number	Per cent	Number	Per cent
Montana	61	45.2	61	45.2	13	9.6	135	100.0
Oregon	241	52.5	171	37.3	47	10.2	459	100.0
Washington	540	53.4	384	37.9	88	8.7	1,012	100.0
Total	842	52.4	616	38.4	148	9.2	1,606	100.0

As shown in Figure 6, the greatest number of accidents occurs during the day, but it is probable that the greatest accident risk is after dark. Figure 7 illustrates the relative amounts of night and day traffic upon the high-

FIG. 6.—Time of accidents in the three States

ways in Oregon during May, 1922, as computed from a traffic census made by the State highway department.[1]

[1] This figure is confirmed very closely by the results shown in "A Report of Traffic on State Highways and County Roads in California, 1922," by the U. S. Bureau of Public Roads and California Highway Commission with the cooperation of 24 California counties.

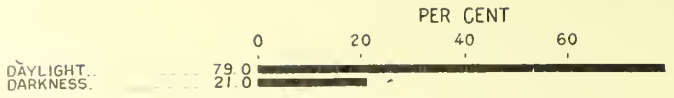

FIG. 7.—Relation of observed day and night traffic in Oregon, May, 1922

Using this information in connection with the present data, Figure 8 has been computed, showing that 2.76 times as many motor vehicles were in accidents after dark as in the daylight. It appears therefore that although the greater number of accidents occurs during

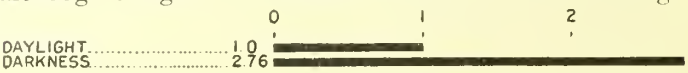

FIG. 8.—Relative number of accidents per motor vehicle in day and night

the day it is probable that the ratio of accidents to the number of motor vehicles on the road is greater at night.

In Figure 3 the summer season is shown as the time of the year during which the greatest number of accidents occurs. It is possible that the greatest number of accidents per 1,000 motor vehicles on the road may occur in the winter months, but no traffic counts are available to confirm this. If 6 per cent of the total annual traffic occurred in December and 15 per cent in July, the accident frequency would be greater during December.[2] It is noticeable from newspaper accounts that the accidents in the Pacific Coast States become frequent at the

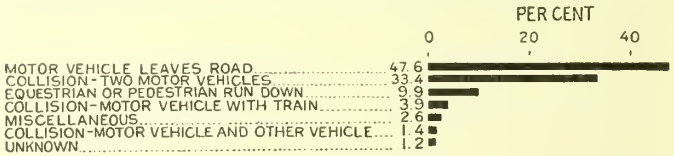

FIG. 9.—Nature of motor-vehicle accidents in the three States, in percentage of total number of accidents

beginning of the rainy season. This may be due to lowered visibility caused by lack of sunlight, to rain or fog in the atmosphere and on the windshield, to use of side curtains, and to skidding, especially on wet or frosty bituminous pavements. A more detailed study will be required to determine during what months the greatest accident risk occurs.

NATURE OF ACCIDENTS

Table 3 and Figure 9 give a summary of the nature of the accidents in the three States over the eight months' period.

TABLE 3.—*Nature of accidents*

Nature of accident	State						Total	
	Montana		Oregon		Washington			
	Number	Per cent	Number	Per cent	Number	Per cent	Number	Per cent
Motor vehicle leaves road	73	54.1	222	48.4	470	46.3	765	47.6
Collision of two motor vehicles	31	23.0	164	35.7	341	33.7	536	33.4
Equestrian or pedestrian run down	8	5.9	31	6.8	120	11.9	159	9.9
Collision of motor vehicle and train	15	11.1	23	5.0	25	2.5	63	3.9
Miscellaneous	5	3.7	6	1.3	30	3.0	41	2.6
Collision of motor vehicle and other vehicle	1	.7	8	1.7	14	1.4	23	1.4
Unknown	2	1.5	5	1.1	12	1.2	19	1.2
Total	135	100.0	459	100.0	1,012	100.0	1,606	100.0

[2] This is a conservative estimate, which is confirmed by the results of the California report and the "Connecticut Highway Transportation Survey," PUBLIC ROADS, vol. 5, No. 1, March, 1924.

The classification "motor vehicle leaves road," means that no collision occurred, although a machine may have swerved from a road to avoid a smash-up or have plunged from the traveled way because of reckless driving, inexperience, intoxication, etc.

LOCATION OF ACCIDENTS

Due to lack of information in the newspaper accounts the location of the accident in more than one-half of the cases is classified as unknown. The balance of the data shown in Table 4 and Figure 10 should be representative, however, of actual conditions.

TABLE 4.—*Location of accidents*

Location of accident	State						Total	
	Montana		Oregon		Washington			
	Number	Per cent	Number	Per cent	Number	Per cent	Number	Per cent
Unknown	74	54.8	248	54.0	640	63.2	962	59.8
On curve	15	11.1	48	10.5	103	10.2	166	10.3
On grade	17	12.6	42	9.2	80	7.9	139	8.7
On straightaway	7	5.2	47	10.2	77	7.6	131	8.2
Intersection of highways	3	2.2	30	6.5	50	4.9	83	5.2
Railroad grade crossing	14	10.4	28	6.1	30	3.0	72	4.5
On bridge	5	3.7	16	3.5	32	3.2	53	3.3
Total	135	100.0	459	100.0	1,012	100.0	1,606	100.0

Contrary to other published data the greatest number of accidents in this section occurs on curves, and the next largest number on grades, which may be on a curve or a straightaway. The number of accidents on the tangents occupies third place. The mountainous character of the country in these three States and the consequent large number of curves may account for the greater number of accidents on the curves. It is conceivable that in the Middle West the greatest number would occur on the straightaways. The grand total curved length of roadway in the State compared with the grand total straightaway length must be an important factor in determining where the greatest number of accidents will occur.

KIND OF ROAD SURFACE IN RELATION TO NUMBER OF ACCIDENTS

What bearing does the type of road have upon the accident rate? This is an important question to which engineers are in need of an answer. Table 5 and Figures 5 and 11 show that the greatest number of accidents occurs on pavements, which represent only 1.9 per cent of the rural road mileage of the three States. But this does not mean that pavements are the most dangerous types *per se*, but that the paved roads are more crowded due to their location near populous centers or on main State arteries, and probably to the higher speed made possible by these types of roads. An attempt was made to determine whether the cement concrete or bituminous pavements were the more dangerous, but was abandoned because of lack of sufficient data to support any positive conclusion. The percentage of accidents attributed to skidding was much higher on the bituminous pavements, but the number of accidents per mile of pavement of each type seems to be practically the same. The crowded condition of the road and speed and recklessness seem to be the chief causes of the accidents, and the increased risk due to a slippery pavement seems to be minimized by additional caution on the part of the drivers.

TABLE 5.—*Number of accidents in relation to type of road surface*

Kind of road surface	State						Total	
	Montana		Oregon		Washington			
	Number	Per cent	Number	Per cent	Number	Per cent	Number	Per cent
Unknown	98	72.6	124	27.0	437	43.2	659	41.1
Pavement	8	5.9	267	58.2	400	39.5	675	42.0
Crushed stone or gravel	22	16.3	57	12.4	151	14.9	230	14.3
Earth	7	5.2	11	2.4	24	2.4	42	2.6
Total	135	100.0	459	100.0	1,012	100.0	1,606	100.0

CAUSES OF ACCIDENTS

The causes of accidents are given in summary form in Table 6 and graphically in Figures 12 and 13. The causes are subdivided into faults of persons, equipment, and highway design.

TABLE 6.—*Summary of the causes of accidents*

	Montana		Oregon		Washington		Total	
	Number	Rate per 1,000 vehicles	Number	Rate per 1,000 vehicles	Number	Rate per 1,000 vehicles	Number	Rate per 1,000 vehicles
Faulty operation by driver:								
Ignorance of traffic rules					3	0.01	3	0.01
Incompetence and inexperience	2	0.03	5	0.03	8	.03	15	.03
Children drivers			1	.01	5	.02	6	.01
Operation by intoxicated persons	4	.06	11	.07	26	.10	41	.08
Excessive speed	21	.31	34	.21	43	.16	98	.20
Recklessness and carelessness	36	.52	197	1.22	478	1.80	711	1.43
Violation of traffic laws	3	.04	18	.11	71	.27	92	.19
Miscellaneous	3	.04	8	.05	43	.16	54	.11
Total	69	1.00	274	1.70	677	2.55	1,020	2.06
Faults of others than driver:								
Child runs across road	1	.01	6	.04	24	.09	31	.06
Pedestrian fails to give right of way			1	.01	5	.02	6	.01
Miscellaneous	19	.28	48	.30	87	.33	154	.31
Total	20	.29	55	.34	116	.44	191	.38
Faulty equipment:								
Poor or improperly adjusted brakes	3	.04	3	.02	10	.04	16	.03
Glaring headlights	3	.04	20	.12	23	.09	46	.09
Driving with only one light	1	.01	3	.02	1		5	.01
Driving without lights			6	.04	6	.02	12	.02
Baggage extending beyond fenders					1		1	
Lack of nonskid appliances			1	.01	31	.12	32	.06
Mechanical breakdowns	21	.30	24	.15	23	.09	68	.14
Miscellaneous	1	.01					1	.01
Total	29	.42	57	.35	95	.36	181	.36

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TABLE 6.—*Summary of the causes of accidents*—Continued

	Montana		Oregon		Washington		Total	
	Number	Rate per 1,000 vehicles	Number	Rate per 1,000 vehicles	Number	Rate per 1,000 vehicles	Number	Rate per 1,000 vehicles
Faulty highway conditions:								
Lack of sight distance					1		1	
Narrow highway	2	0.03	8	0.05	9	0.03	19	0.03
Narrow bridge or culvert			2	.01	1		3	.01
Inadequate, irregular, or improper signs					1		1	
Skiddy surface	8	.12	51	.32	91	.34	150	.30
Lack of guardrail			1	.01			1	
Unnecessary obstruction of view	1	.01	2	.01	5	.02	8	.02
Holes or depressions in highway			2	.01	4	.02	6	.01
Miscellaneous	6	.09	7	.04	12	.05	25	.05
Total	17	.25	73	.45	124	.47	214	.43
SUMMARY								
Faulty operation by driver	69	1.00	274	1.69	677	2.55	1,020	2.06
Faults of others than drivers	20	.29	55	.34	116	.44	191	.38
Faulty equipment	29	.42	57	.35	95	.36	181	.36
Faulty highway conditions	17	.25	73	.45	124	.47	214	.43
Grand total	135	1.95	459	2.84	1,012	3.81	1,606	3.23

TABLE 7.—*Detailed analysis of accidents in Montana*

	1923	1924							Total	
	December	January	April	May	June	July	August	September	Number	Per cent
Number of accidents	14	5	14	13	19	14	38	18	135	100.0
Results:										
Number of fatalities	3	3	2	1	5	4	12	3	33	24.4
Number of persons injured	17	3	16	17	30	27	42	16	168	124.4
Number of motor vehicles damaged	14	2	11	11	15	11	25	14	103	76.2
Time:										
Daylight	5	3	9	8	9	8	14	5	61	45.2
Darkness	7	2	5	5	9	6	16	11	61	45.2
Unknown	2				1		8	2	13	9.6
Total	14	5	14	13	19	14	38	18	135	100.0
Nature of accident:										
Collision of two motor vehicles	3		3	3	7	3	8	4	31	23.0
Collision of motor vehicle with other vehicle					1				1	.7
Collision of motor vehicle and train	5	1	1			1	6	1	15	11.1
Motor vehicle leaves road	4	1	9	9	9	9	21	11	73	54.1
Equestrian or pedestrian run down	1	2	1		2		2		8	5.9
Miscellaneous		1		1			1	2	5	3.7
Unknown	1					1			2	1.5
Total	14	5	14	13	19	14	38	18	135	100.0
Location:										
Railroad grade crossing	4	1	1			1	6	1	14	10.4
Intersection of highways			1				2		3	2.2
On straightaway	5	1	1						7	5.2
On curve	2	1		3	1	1	4	3	15	11.1
On grade	1		2	3	2	2	6	1	17	12.6
On bridge			1	2			2		5	3.7
Unknown	2	2	8	5	16	10	18	13	74	54.8
Total	14	5	14	13	19	14	38	18	135	100.0
Kind of road surface:										
Earth				1	1	1	3	1	7	5.2
Macadam, gravel, or crushed rock	1				2	10	6	3	22	16.3
Pavement			2		2	1	1	2	8	5.9
Unknown	13	5	12	12	14	2	28	12	98	72.6
Total	14	5	14	13	19	14	38	18	135	100.0
CAUSES										
Faulty operation by driver:										
Incompetence and inexperience			1				1		2	1.5
Operation by intoxicated persons			2	2					4	3.0
Excessive speed	1	1	1	5	9	1	2	1	21	15.6
Recklessness and carelessness	10	2	5			1	13	5	36	26.8
Violation of traffic law						2	1		3	2.2
Miscellaneous			1			2			3	2.2
Total	11	3	10	7	9	6	17	6	69	51.3
Faults of others than driver:										
Child runs across road							1		1	.7
Miscellaneous	2	1			1	1	8	6	19	14.1
Total	2	1			1	1	9	6	20	14.8
Faulty equipment:										
Poor or improperly adjusted brakes							3		3	2.2
Glaring headlights	1			1				1	3	2.2
Driving with only one light					1				1	.7
Mechanical breakdowns		1	3	3	4	4	1	5	21	15.6
Miscellaneous							1		1	.7
Total	1	1	3	4	5	4	5	6	29	21.4
Faulty highway conditions:										
Narrow highway				1			1		2	1.5
Unnecessary obstruction of view							1		1	.7
Skiddy surface					2	3	3		8	5.9
Miscellaneous			1	1	2		2		6	4.4
Total			1	2	4	3	7		17	12.5
SUMMARY OF CAUSES										
Faulty operation by driver	11	3	10	7	9	6	17	6	69	51.3
Faults of others than driver	2	1			1	1	9	6	20	14.8
Faulty equipment	1	1	3	4	5	4	5	6	29	21.4
Faulty highway conditions			1	2	4	3	7		17	12.5
Grand total	14	5	14	13	19	14	38	18	135	100.0

TABLE 8.—*Detailed analysis of accidents in Oregon*

	1923	1924							Total	
	December	January	April	May	June	July	August	September	Number	Per cent
Number of accidents	41	39	37	65	45	101	73	58	459	100.0
Results:										
Number of fatalities	2	3	10	10	6	8	15	2	56	12.2
Number of persons injured	40	45	29	61	56	93	70	54	448	97.6
Number of motor vehicles damaged	42	44	34	48	36	78	62	42	386	84.1
Time:										
Daylight	17	25	24	32	25	48	35	35	241	52.5
Darkness	22	13	12	32	16	37	26	13	171	37.3
Unknown	2	1	1	1	4	16	12	10	47	10.2
Total	41	39	37	65	45	101	73	58	459	100.0
Nature of accident:										
Collision of two motor vehicles	17	14	13	25	12	34	34	15	164	35.7
Collision of motor vehicle with other vehicle		1	1		2	1	2	1	8	1.7
Collision of motor vehicle and train	3	3	3	6	1	2		5	23	5.0
Motor vehicle leaves road	15	18	16	33	24	54	31	31	222	48.4
Equestrian or pedestrian run down	5	3	2	1	5	7	5	3	31	6.8
Miscellaneous					1	2	1	2	6	1.3
Unknown	1		2			1		1	5	1.1
Total	41	39	37	65	45	101	73	58	459	100.0
Location:										
Railroad grade crossing	3	4	5	5	2	3	1	5	28	6.1
Intersection of highways	5	2	3	3	3	4	6	4	30	6.5
On straight away	12	7	4	10		3	6	5	47	10.2
On curve	5	3		10	6	11	5	8	48	10.5
On grade	4	2	2	5	4	9	7	9	42	9.2
On bridge		2	4	1	2	4	3		16	3.5
Unknown	12	19	19	31	28	67	45	27	248	54.0
Total	41	39	37	65	45	101	73	58	459	100.0
Kind of road surface:										
Earth				1	1	4	5		11	2.4
Macadam, gravel, or crushed rock	9		7	9	6	20	4	2	57	12.4
Pavement	25	39	21	41	24	52	35	30	267	58.2
Unknown	7		9	14	14	25	29	26	124	27.0
Total	41	39	37	65	45	101	73	58	459	100.0
CAUSES										
Faulty operation by driver:										
Incompetence and inexperience	3			1			1		5	1.1
Children drivers				1					1	.2
Operation by intoxicated persons	3		2	2	1	1	1	1	11	2.4
Excessive speed	2	2	1	4	7	4	10	4	34	7.4
Recklessness and carelessness	21	12	15	27	15	45	34	28	197	43.0
Violation of traffic law	3	1		4		8		2	18	3.9
Miscellaneous		3	1		2	2			8	1.7
Total	32	18	19	39	25	60	46	35	274	59.7
Faults of others than driver:										
Child runs across road		2	1			1	1	1	6	1.3
Pedestrian fails to give right of way							1		1	.2
Miscellaneous	1	1	8	7	5	14	8	4	48	10.6
Total	1	3	9	7	5	15	10	5	55	12.1
Faulty equipment:										
Poor or improperly adjusted brakes						1	1	1	3	.7
Glaring headlights	5	2	1		3	3	2	4	20	4.4
Driving with only one light				1		2			3	.7
Driving without lights	2	2				1	1		6	1.3
Lack of nonskid appliances			1						1	.2
Mechanical breakdowns	1			3	4	7	2	7	24	5.2
Total	8	4	2	4	7	14	6	12	57	12.5
Faulty highway conditions:										
Narrow highway			1		1	5		1	8	1.7
Narrow bridge or culvert			1	1					2	.4
Lack of guardrail			1						1	.2
Unnecessary obstruction of view							1	1	2	.4
Holes or depressions in highway				1				1	2	.4
Skiddy surface		14	4	12	4	5	9	3	51	11.1
Miscellaneous				1	3	2	1		7	1.5
Total		14	7	15	8	12	11	6	73	15.7
SUMMARY OF CAUSES										
Faulty operation by driver	32	18	19	39	25	60	46	35	274	59.7
Faults of others than driver	1	3	9	7	5	15	10	5	55	12.1
Faulty equipment	8	4	2	4	7	14	6	12	57	12.5
Faulty highway conditions		14	7	15	8	12	11	6	73	15.7
Grand total	41	39	37	65	45	101	73	58	459	100.0

TABLE 9.—*Detailed analysis of accidents in Washington*

	1923	1924							Total	
	December	January	April	May	June	July	August	September	Number	Per cent
Number of accidents	69	55	44	152	128	194	215	155	1,012	100.0
Results:										
Number of fatalities	8	7	12	12	24	11	17	13	104	10.3
Number of persons injured	75	36	92	158	196	159	187	132	1,035	102.2
Number of motor vehicles damaged	67	57	103	111	146	147	168	133	932	92.1
Time:										
Daylight	32	27	28	89	70	106	109	79	540	53.4
Darkness	31	23	16	54	47	75	78	60	384	37.9
Unknown	6	5		9	11	13	28	16	88	8.7
Total	69	55	44	152	128	194	215	155	1,012	100.0
Nature of accident:										
Collision of two motor vehicles	25	17	16	45	41	64	73	60	341	33.7
Collision of motor vehicle with other vehicle	1			1	3	2	3	4	14	1.4
Collision of motor vehicle and train	1	1	1	6	2	5	4	5	25	2.5
Motor vehicle leaves road	30	25	17	68	62	99	105	64	470	46.3
Equestrian or pedestrian run down	9	11	10	23	15	17	18	17	120	11.9
Miscellaneous		1		3	5	7	10	4	30	3.0
Unknown	3			6			2	1	12	1.2
Total	69	55	44	152	128	194	215	155	1,012	100.0
Location:										
Railroad grade crossing	2	1	1	9	2	5	5	5	30	3.0
Intersection of highways	5	2	3	8	6	6	9	11	50	4.9
On straightaway	12	4	4	8	2	14	17	16	77	7.6
On curve	10	2	2	13	12	27	24	13	103	10.2
On grade	8	9	3	6	12	15	18	9	80	7.9
On bridge		3		5	2	10	10	2	32	3.2
Unknown	32	34	31	103	92	117	132	99	640	63.2
Total	69	55	44	152	128	194	215	155	1,012	100.0
Kind of road surface:										
Earth	1			1	2	7	9	4	24	2.4
Macadam, gravel, or crushed rock	4	3	3	24	14	48	33	22	151	14.9
Pavement	18	41	20	41	35	86	85	74	400	39.5
Unknown	46	11	21	86	77	53	88	55	437	43.2
Total	69	55	44	152	128	194	215	155	1,012	100.0
CAUSES										
Faulty operation by driver:										
Ignorance of traffic rules					3				3	.3
Incompetence and inexperience	2		2		4				8	.8
Children drivers								5	5	.5
Operation by intoxicated persons	3	1	3		6	8	5		26	2.6
Excessive speed	3		4	8	7	8	6	7	43	4.2
Recklessness and carelessness	41	19	16	65	63	97	102	75	478	47.1
Violation of traffic laws	10	4	2	12	7	19	9	8	71	7.0
Miscellaneous		1	1	1	7	26	6	1	43	4.3
Total	59	25	28	86	97	158	128	96	677	66.8
Faults of others than driver:										
Child runs across road		2	2	3	2	2	1	12	24	2.4
Pedestrian fails to give right of way	1	1	1		2				5	.5
Miscellaneous	4	4	6	27	6		24	16	87	8.5
Total	5	7	9	30	10	2	25	28	116	11.4
Faulty equipment:										
Poor or improperly adjusted brakes				4	2	1	3		10	1.0
Glaring headlights	4	4	1	3	2	1	5	3	23	2.3
Driving with only one light								1	1	.1
Driving without lights		1	1	1			3		6	.6
Baggage extending beyond fenders							1		1	.1
Lack of nonskid appliances				1		11	13	6	31	3.1
Mechanical breakdowns	1	1	2	14	5				23	2.3
Total	5	6	4	23	9	13	25	10	95	9.5
Faulty highway conditions:										
Lack of sight distance		1							1	.1
Narrow highway				1		5		3	9	.9
Narrow bridge or culvert				1					1	.1
Inadequate, irregular, or improper signs		1							1	.1
Unnecessary obstruction of view			2	1	1			1	5	.5
Holes or depressions in highway		3			1				4	.4
Skiddy surface		11	1	6	10	14	32	17	91	9.0
Miscellaneous		1		4	1	1	5		12	1.2
Total		17	3	13	12	21	37	21	124	12.3
SUMMARY OF CAUSES										
Faulty operation by driver	59	25	28	86	97	158	128	96	677	66.8
Faults of others than driver	5	7	9	30	10	2	25	28	116	11.4
Faulty equipment	5	6	4	23	9	13	25	10	95	9.5
Faulty highway conditions		17	3	31	12	21	37	21	124	12.3
Grand total	69	55	44	152	128	194	215	155	1,012	100.0

HIGHWAY SPEED ZONES SUGGESTED

Making the highways safe for the general public should be one of the chief functions of the highway engineer in the development of improved motor transport. The whole problem of regulation has hitherto been too generally regarded as a function of the police authorities, rather than as an engineering and economic problem, the end of which is the provision of safe and adequate highway transportation. Since the appearance of the motor vehicle, public opinion and the consequent attitude of government toward its use have

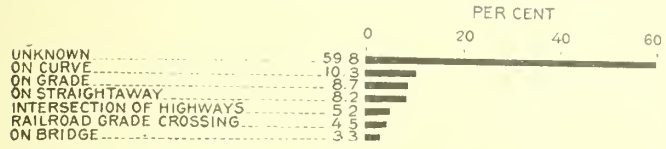

FIG. 10.—Location of accidents in percentage of the total number

passed through two stages. In the first stage, corresponding to the period when motor vehicles were few in number and there was abundant room for them on the roads, they were regarded as a life-menacing luxury, and inflexible and unnecessarily low maximum speed limits were rigidly enforced. In the second stage, which coincides with the increase in motor-vehicle registration to large proportions, the roads have become crowded and at the same time there has been a softening of the rigors of speed regulation, largely the result of growing confidence in the motor vehicle and the overcoming of the fears and jealousies which it inspired in the early days.

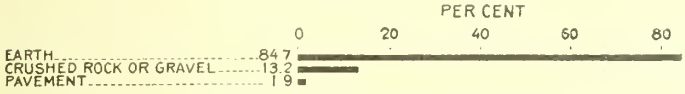

FIG. 11.—Classification of roads, by type, in the three States

It may now be timely to suggest that this liberalizing tendency, desirable as it is in the interest of free transportation, has proceeded "not wisely but too well." Whereas it is certainly essential to impose no more severe restrictions on the operation of vehicles than may be absolutely necessary, it is also important that liberality be kept within the bounds of safety.

The statistics presented in this article clearly indicate that the greater risk exists on crowded highways. It

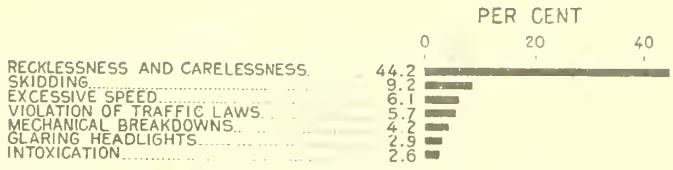

FIG. 12.—Chief causes of accidents in the three States, in percentage of the total number

follows therefore that on such highways there is need of greater care in operation and the adoption of whatever measures may lend themselves to the reduction of the seriousness of unavoidable accidents. While the prevention of recklessness, as already suggested, is difficult of attainment short of the elimination of the reckless driver—and how many are not at times reckless—the reduction of speed on congested roads will go far to prevent the occurrence of the more serious accidents.

Ultimately there will be need undoubtedly for a reduction of congestion by provision of more extensive highway facilities. The part the highway engineer may play in this solution is illustrated by the proposed grade separation and superhighway now planned from Detroit to Pontiac, Mich. The Washington State Highway Department is also preparing to reduce the traffic

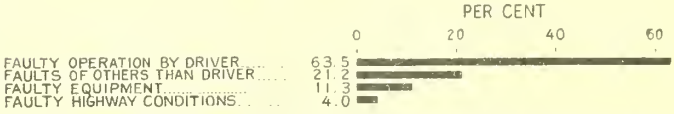

FIG. 13.—Summary of the causes of accidents in the three States, in percentage of the total number

density between Seattle and Tacoma by the construction of two 18-foot parallel pavements separated by a parking space, all built over an entirely new route. Each side of the road will be used by traffic moving in one direction only. But such engineering solutions of the problem will take some time to accomplish, and there is need of immediate relief.

It is believed that the establishment of speed zones, in each of which the maximum speed limit will be determined by the congestion of traffic in the zone, will in some measure provide an immediate remedy. Such a plan is now in effect in Maryland, apparently with favorable results.

CRUSHED-STONE TESTS AND THEIR RELATION TO THE SERVICE OF THE FINISHED PAVEMENT

By A. T. GOLDBECK, Chief Division of Tests, Bureau of Public Roads

THE TREND in the writing of present-day highway specifications is toward definiteness with respect to every item involved. Materials are described in as much detail as our knowledge seems to warrant. It has been found that much contention on the part of the engineer, the contractor, and the materials producer is saved when each one knows definitely what is expected of him. Specifications for materials involve a description of the materials in terms of specific qualities which those materials possess in order to be satisfactory for the particular type of construction contemplated and it is generally the case that where there is much deviation from the specifications, unsatisfactory results are likely to occur.

In general, there are two properties which crushed stone must possess for its successful application to any particular type of road construction: (1) It must be crushed to the proper size, and (2) it must have physical qualities which make for enduring construction.

Crushed stone was first used to a considerable extent in the building of the macadam type of road in the days of steel-tired traffic. As early as 1878 the first physical test was developed in the French School of Bridges and Roads, primarily as a test of the suitability of stone for waterbound macadam construction. Later the Dorry test for hardness of rock was likewise developed in France, again primarily to determine the suitability of rock for waterbound macadam. In this country other tests were developed by the late Logan Waller Page in the then Office of Public Roads, United States Department of Agriculture. These tests were the Page impact test for determining the toughness of

rock, the cementing value test to determine the binding value of rock dust, and the absorption test. Lately a new test has been developed, but not perfected, the so-called accelerated soundness test for rock.

PHYSICAL TESTS FOR ROCK

The Deval abrasion test.—The Deval abrasion test is made by placing a sample of 50 pieces of uniform-sized stone in a cylinder inclined at an axis. As the cylinder is revolved the pieces of rock and particularly their corners are worn to dust, and the amount of wear is expressed in percentage of the original sample. The French coefficient of wear equals 40 divided by the percentage of wear. Both terms are used and should not be confused.

The Dorry hardness test.—In the Dorry hardness test a cylinder of rock is held against a revolving disk upon which dry crushed quartz is fed, and the amount of wear is determined.

The toughness test.—In the toughness test a 1-inch cylinder of rock is subjected to the impact of a hammer falling from an increasing height on a round-ended plunger resting on the specimen. The toughness of the rock is expressed in terms of the height of fall of the hammer when failure takes place.

Cementing value test.—In the cementing value test the rock is ground to powder and mixed with water in a ball mill, then compressed into a cylinder, which is dried in an oven and finally broken in a special impact machine, whose hammer falls from a constant height of 1 centimeter until the specimen fails. The number of blows required to produce failure indicates the relative cementing value of the rock.

The absorption test.—The absorption test consists simply of determining the amount of water a rock will absorb in a given time.

All of these tests were designed in the days of water-bound macadam, and they have been criticized as being unsuitable in their application to our present-day uses of rock in Portland cement concrete, bituminous macadam, and bituminous concrete pavements subjected to rubber-tired traffic. A discussion of this point will not be amiss at this time. In the days of waterbound macadam and steel tires the rock was subjected to surface abrasion by vehicles, which created a considerable amount of dust, and also to internal wear, due to the grinding of one rock on another. The dust created served to bind the road together, the dust of some rocks possessing this property more than others.

This was the sole reason for the development of the cementing value test. At the present time it has practically no significance for we can no longer depend on the cementing value of the dust to hold the road together. Under rubber-tired traffic very little dust is formed by abrasion and this is rapidly dissipated. There is little excuse then for the cementing value test in present-day specifications for crushed rock.

In bituminous macadam of the penetration type the action of rubber-tired traffic still produces internal wear particularly of the stone not covered with bituminous material. There is still need therefore for some kind of abrasion test such as the Deval test. In the bituminous concrete type of pavement the surface stone is still subjected to forces of high intensity such as impact from high-speed traffic and the action of tire chains and it is likewise our belief that there is the possibility of considerable internal wear when the stone is too soft. It will be seen therefore that there is still need for tests to determine the strength of the rock. The Deval test and the toughness test both accomplish this purpose.

In concrete pavements the stone is surrounded by Portland cement mortar, which prevents any movement and which offers considerable protection. The stone is subjected only to the abrasive action of traffic and to a slight extent to the disintegrating influence of the weather. It has been found both in service and by means of specially conducted tests that when the stone in a concrete pavement is so soft that it wears faster than the mortar, uneven and rapid wear results, especially where tire chains are used. We need have no fear of abrasive wear from rubber-tired traffic alone. Some of these old-fashioned tests then, such as the cementing value test and Dorry hardness test, no longer have any particular significance when applied to crushed stone as now used in highway construction.

ABRASION AND TOUGHNESS TESTS STILL VALUABLE

On the other hand, since stone under present-day traffic conditions must still possess strength, durability, and resistance to abrasion the tests which measure these properties are still applicable. The Deval abrasion test and the toughness test for rock still have high value, though undoubtedly they might be improved upon. It is for the above reasons that highway specifications for rock still contain clauses which require a specified maximum percentage of wear or a certain minimum toughness. The toughness test has been largely supplanted by the more simple Deval test in many laboratories, for it has been found in general that the Deval test serves every purpose. It is a toughness test and an abrasion test combined. Some specifications further safeguard the quality of the rock through the use of both the toughness and the Deval test.

I have mentioned two other tests, the absorption and the soundness tests. The absorption test measures the amount of water which is absorbed by the rock. Ordinarily rocks which have a high absorption are soft and are not apt to withstand constant repetition of frost action. The accelerated soundness test is a test which is designed to simulate the destructive action of repeated freezing and thawing. As at present performed the rock is immersed in a saturated solution of sodium sulphate for 20 hours, dried for 4 hours in an oven, and this treatment repeated five times. Rocks which are not durable under frost action will disintegrate or crack under this test. Sodium sulphate crystallizes in the pores of the rock, gradually expanding and destroying the structure much after the action of ice. The indications are that the test is too severe. A rock which fails in the test will not necessarily fail in service, but apparently the test does detect rocks which are likely to fail under service.

TEST LIMITS FOR VARIOUS TYPES OF CONSTRUCTION

Problems in economics are involved in the use of materials and for this reason specifications for materials to be used in similar types of construction differ in different States. In some localities it is advisable to use material which in other States is regarded as inferior, for ultimate economy results even though the road does not possess the highest lasting qualities. Crushed stone is now used for the most part in macadam

construction which will later receive a surface treatment of bituminous material, in bituminous macadam, in bituminous concrete, and Portland cement concrete pavements.

Generally where waterbound macadam is built, to be later surface-treated, heavy traffic is not expected. Here good practice would not as a rule allow stone with a percentage of wear of more than 6. Economy may demand, however, that a softer stone be used in certain localities. For bituminous macadam a percentage of wear of more than 6 should not be used; for bituminous concrete, which ordinarily is of a more expensive type laid on a more expensive base than bituminous macadam, the percentage of wear should not be more than 5, and in order to reduce the liability of the stone to fracture, an additional requirement for toughness is used, the value for toughness being set at not less than 6. Instances are known where disintegration of bituminous concrete seems to have resulted from the use of too soft a stone. Many engineers specify a lower percentage of wear for trap or granite than for limestone for a given type of construction. The reason for this is that trap and granite are naturally more resistant than limestone, so that granite, showing a percentage of wear greater than normal for that type, is likely to be partially disintegrated or of otherwise inferior quality even though the actual wear as shown by the tests is the same as the limestone. In connection with waterbound macadam construction certain types of rock, such as quartzite, gneiss, and schist, generally can not be used successfully, due to the difficulty of binding such materials.

In order to study the physical properties of stone desirable for concrete road construction, the Bureau of Public Roads has conducted a rather elaborate test in which 62 different kinds of concrete containing imported aggregates were subjected to the abrasive wear of a special rubber-tired vehicle operated at 20 miles an hour. The vehicle was guided by means of a track so that the wheels ran continuously in the same track. In this way the test was accelerated. The vehicle was first equipped merely with solid tires, and after running it for thousands of trips it was concluded that, so far as rubber-tired traffic alone is concerned, the present wear on concrete roads is not an important factor in limiting the life of the road.

In many sections of the country, however, there are conditions which make for abrasive wear, such as the use of tire chains, and grooves have been worn in concrete pavements due to this cause. The test vehicle was therefore equipped with tire chains and the test repeated and soon the surface wear became very appreciable. This test has been described in detail in PUBLIC ROADS, and the important points brought out were:

> That rubber-tired traffic alone produces no appreciable wear on a concrete pavement.
>
> That tires equipped with tire chains cause appreciable wear. This wear is practically independent of the hardness or type of coarse aggregate so long as the coarse aggregate has at least the same resistance to wear as the mortar. When softer aggregates than these are used, the wear of the pavement increases as the stone becomes softer. So far as can be determined the percentage of wear in the Deval abrasion test, which results in equal wear in both stone and mortar, is approximately 7.0.
>
> From the standpoint of abrasive wear alone there seems to have been little choice in this test between crushed stone and gravel. It should be emphasized, however, that abrasive wear is but one of several factors which should influence the choice of an aggregate. The question of structural strength of the concrete, relative liability to cracking, and subsequent spalling and consequently high maintenance expense must also be considered. It appears from these tests that notwithstanding the fact that the Deval abrasion test was originally designed for macadam road construction it seems to have considerable significance for the specification of rock for concrete road construction.

USE OF SCREENINGS AS FINE AGGREGATE

While discussing the question of physical properties of coarse aggregate for concrete roads, it will be well to consider the use of stone screenings as a fine aggregate in concrete. In general stone screenings are characterized by the presence of an excessively large amount of dust and an excessively large amount of very coarse particles with a corresponding lack of particles of intermediate size when compared with the grading of good concrete sand. One of the tests for the suitability of fine aggregate for use in concrete is the so-called strength-ratio test in which the tensile strength of 1:3 mortar briquets is compared with the tensile strength of Ottawa sand mortar briquets of the same proportions made at the same time. The ordinary assumption is that when these strengths are equal the sand is an excellent concrete material and in general this holds true. When this test is made with stone screenings, it is almost invariably found that the briquets made with the screenings test higher than Ottawa sand briquets, but notwithstanding this fact, it is generally true that concrete made with stone screenings as the fine aggregate has a lower strength than concrete in which sand is used. It is also true that in many cases where stone screenings have been used the concrete has not been resistant to the weather. Particular trouble has been had with limestone screenings, and no doubt this is because the concrete has been lacking in density, due to the poor grading of the screenings. If it were possible for crushed-stone producers to turn out screenings with the same mechanical grading as a good grade of concrete sand there would be no question of its suitability for use

UNIFORMITY OF CONCRETE

The technical literature of late has placed emphasis on the question of uniformity of concrete. Concrete structures, including highways, are proportioned on the assumption that the concrete has a definite assumed strength. Thousands of concrete cores have been drilled from concrete roads, and the results in many cases show a great lack of uniformity in the concrete, the crushing strength in some cases varying as much as 100 per cent on the same job. It is not sufficient that the concrete shall have an average strength equal to that for which the structure was proportioned when it is so much weaker than that in many spots. It is highly important that everything possible be done to bring about uniformity of strength. Already several of the State highway departments have begun to take measures aiming at more uniform concrete. Iowa is now carefully weighing both sand and stone entering into concrete road construction. Other States although still measuring by volume, are taking into account the fact that the volume of sand may vary 25 per cent due to variable moisture content. Devices are beginning to appear which aim not only to insure more exact measurement of the fine and coarse aggregate, but also of the water used for mixing, which is another of the decided variables which result in lack of uniformity.

There are few engineers who realize the part the coarse aggregate plays in influencing the water content in a concrete mixture and the strength of the resulting concrete. Whenever the gradation of the coarse aggregate changes during the progress of the work, this necessitates changes in the amount of water required to produce a given consistency of concrete.

To illustrate the effect of gradation on consistency, tests were made on 1:2:3 concrete, using both limestone and gravel as the coarse aggregate. The results are indicated in Table 1. It will be observed that changes in the gradation of the coarse aggregated produced a marked effect on the workability. For instance, a certain amount of water produced a slump of 3 inches when a ¼ to 2 inch normally graded aggregate was used, whereas exactly the same quantity of water produced a slump of 6 inches when the material between the ¼ and 1 inch screens was removed. On the other hand, by maintaining a constant consistency, which of course should be the aim of every mixer operator, fully ½ gallon less of water per bag of cement was required when the stone was graded from 1 to 2 inches than when normally graded from ¼ to 2 inches. In a 6-bag batch this would mean a change of 3 gallons in the measured water content and of course a correspondingly large difference in strength.

TABLE 1.—*Results of tests of consistency of 1 : 2 : 3 concrete, varying the size of aggregate and amount of mixing water*

Grading of aggregate	Crushed stone		Gravel	
	Water required per bag of cement for a given consistency	Slump when a constant amount of water is used	Water required per bag of cement for a given consistency	Slump when a constant amount of water is used
Inches	*Gallons*	*Inches*	*Gallons*	*Inches*
¼–¾	8.0	1	4.5	2
¼–1	7.7	2	4.4	3
¼–2	7.3	3	4.1	3
¾–2	7.0	4	3.9	6
1–2	6.8	6	3.8	7

Producers of stone can aid in the movement for more uniform concrete by maintaining an absolutely uniform product during the progress of the work. This is an ideal which at the present time is not being generally attained. No doubt it is due in part to the present custom of screening aggregates at the producing plant and then more or less inefficiently mixing the various sizes together in a car before shipment. Possibly if more accurate methods were used in combining the screened sizes the desired results would be obtained. One possible solution for obtaining more uniformly graded coarse aggregate is that of shipping the screened sizes separately to the central proportioning plant, stock-piling them there and later combining them in the desired proportions. It is recognized, however, as one of the difficulties in this proposal that ordinarily it is hard to find sufficient space to stockpile separate sizes, and of course additional time is required to measure the sizes separately at the proportioning plant.

SIZE REQUIREMENTS

In addition to quality requirements for crushed stone, specifications call for grading of the stone in a particular manner for specific types of construction. Unfortunately engineers in different States have different conceptions of what constitutes a proper size of material for the same kind of work, and this of course leads to needless expense in the producing plant which is called upon to supply materials to several adjacent States. It would be highly economical if a definite standard of sizes could be set up which might be followed by the users of crushed stone. The road materials committee of the American Society for Testing Materials is now at work on the development of such a standard. The standard of sizes proposed by this committee which is designed to meet the requirements of the various types of constructions now used is given in Table 2.

TABLE 2.—*Maximum permissible range in mechanical analysis of stone for nominal sizes as proposed by the road materials committee of the American Society for Testing Materials*

Designated size	Percentage by weight passing various sizes of laboratory screens									
	¼ inch	½ inch	¾ inch	1 inch	1¼ inches	1½ inches	2 inches	2½ inches	3 inches	3½ inches
Inches	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
0–¼ [1]	85–100	100	-------	-------	-------	-------	-------	-------	-------	-------
0–½ [1]	15–75	95–100	100	-------	-------	-------	-------	-------	-------	-------
0–¾ [1]	15–75	-------	95–100	100	-------	-------	-------	-------	-------	-------
¼–½	0–15	95–100	100	-------	-------	-------	-------	-------	-------	-------
¼–¾	0–15	25–75	95–100	100	-------	-------	-------	-------	-------	-------
¼–1¼	0–5	-------	40–75	-------	95–100	100	-------	-------	-------	-------
¼–2	0–5	5–25	-------	40–75	-------	-------	95–100	100	-------	-------
¼–2½	0–5	-------	10–25	-------	40–75	-------	-------	95–100	100	-------
½–1	0	0–15	25–75	95–100	100	-------	-------	-------	-------	-------
¾–1¼	-------	-------	0–15	25–75	95–100	100	-------	-------	-------	-------
1¼–2½	-------	-------	-------	-------	0–15	-------	25–75	95–100	100	-------
2½–3½ [2]	-------	-------	-------	-------	-------	-------	-------	0–15	25–75	95–100

[1] Designated sizes 0–¼, 0–½, and 0–¾ when used as screenings in waterbound macadam road construction shall conform to the following additional requirement: Passing 100-mesh sieve, 6 to 12 per cent.

[2] The lower limit for designated size 2½–3½ may be changed to 2 inches when necessary to utilize the product of a crushed producing designated size ¼–2 inches. The upper limit for designated size 2½–3½ may be changed to 4 inches in the case of very soft stone or light or porous slag.

It will be observed that the sizes are stated in terms of laboratory screens. This procedure is followed rather than that of stating the size in terms of the size of the commercial revolving screens simply because there are many factors in plant practice which affect the size of the screened stone, whereas the laboratory practice is standardized. It is the individual producer's problem to run his plant in such a manner that the laboratory screen requirements will be met. There is no doubt that the various laboratories will be glad to work with producers to this end for procuring the desired result.

The various sizes of crushed stone and crushed slag recommended by the road materials committee as shown in Table 2 are thought to be adapted for the following uses:

Nominal size	Intended use
Inches	
0–¼	Fine screenings for waterbound road construction.
0–½	Aggregate for fine-graded bituminous concrete.
0–¾	Coarse screenings for waterbound macadam road construction or aggregate for bituminous concrete.
¼–½	Fine dustless screenings for bituminous road construction.
¼–¾	Coarse dustless screenings for bituminous road construction; coarse aggregate for cement concrete where the maximum size is limited to ¾ inch.
¼–1¼	Coarse aggregate for coarse-graded bituminous concrete; coarse aggregate for cement concrete where the maximum size is limited to 1¼ inches.
¼–2	Coarse aggregate for cement concrete pavements or other concrete structures where the maximum size is limited to 2 inches.
¼–2½	Coarse aggregate for cement concrete pavements or other concrete structures where the maximum size is limited to 2½ inches; coarse aggregate for bituminous concrete base.
½–1	Commercial ¾-inch stone for bituminous road construction.
¾–1¼	Commercial 1-inch stone for bituminous road construction; binder stone for sheet asphalt, etc.
1¼–2½	Coarse aggregate for bituminous macadam, penetration method, and bituminous macadam base; wearing course for waterbound macadam.
2½–3½	Coarse aggregate for bituminous macadam base or base course for waterbound macadam.

REINFORCED CONCRETE PAVEMENT SURVEY

HIGHWAY RESEARCH BOARD REPORTS PROGRESS OF ITS STUDY

By C. A. HOGENTOGLER, Highway Research Board, National Research Council

THE STATUS of the survey for determining the economic value of steel reinforcement in concrete pavements carried on by the highway research board of the National Research Council is such that the procurement of very definite information on this much-discussed question seems assured. As in every other research, it was necessary to have specimens from which were eliminated all variables except those whose effects were to be studied. Instead of being especially constructed, as is ordinarily the case, in this instance they had to be selected from the vast mileage of plain and reinforced concrete roads which has been constructed up to the present time. Since the difficulties attendant upon the attempt to compare a road of one type with another of a different type were keenly appreciated, every effort was made to procure for study only such roads as contained both plain and reinforced sections. The seeking out of such specimens has taken considerable time, since, unfortunately for the purpose, roads are generally either entirely plain or entirely reinforced.

It is now felt that the time and efforts spent in the selection of the desired roads have been amply rewarded, since they have been found in sufficient number to warrant conclusive results. In addition to such experimental service roads as the DeKalb, Ill., and the Sacramento-Riverbank, Calif., surfaces, constructed in 1912; the Du Pont highway in Delaware, constructed in 1915; the Milwaukee County, Wis., and the Los Alamos-Gaviota, Calif., roads, constructed in 1917; and the Columbia Pike, Va., constructed in 1921, there have been located 188 highways whose surfaces contain both plain and reinforced concrete sections which were built at the same time, by the same contractor, under the same supervision, and from the same materials and have been subjected to the same traffic and climatic conditions. Thus in these roads all undesirable variables are eliminated except those of subgrade and strength of concrete, which are common to all highway experiments. These highways are distributed through 30 States, ranging from Connecticut to California and from Minnesota to Texas, with one in Ontario, Canada. In three counties of New York State alone, representing both good and poor subgrade subjected to extreme frost action, there are 304 sections containing reenforcement, of lengths varying from 100 to 1,000 feet, with a total length of over 200,000 lineal feet, each section being part of a concrete road and having plain sections on each side. Contrasted with this soil condition is one found in Mississippi, where, although frost is lacking, the soil has an extremely high volumetric change. In this location an experimental road 1 mile long, half of which was reinforced, was constructed in 1915.

In addition to differences in subgrade conditions these roads represent age, mix, traffic, and cross-section variations. In the reinforcement are included different weights of mesh and various designs using bars.

The final road inspections are now in progress, and will probably continue throughout the summer, after which the data will be analyzed and a report of the findings submitted. Analyses of subgrade samples taken in connection with the investigation are being made by the United States Bureau of Public Roads. The University of Maryland is cooperating in the examination of cores for determination of the effect of time and cracks on reinforcement; and various State highway departments have offered cooperation in the road inspections and core-drill work for checking location of steel and thickness of slab.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JANUARY 31, 1925

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4 598 721.63	$ 2 186 247.64	464.1	$ 844 187.95	$ 418 267.76	87.1	$ 16 357 875.48	$ 7 448 337.90	837.9	$ 79 416.15	$ 39 708.07	0.6	$ 1 160 401.73	Alabama
Arizona	8 338 365.41	4 287 683.88	527.8	713 173.87	406 825.70	37.5	1 992 348.05	1 208 209.81	169.2	656 410.72	407 243.56	51.6	1 185 738.06	Arizona
Arkansas	11 094 751.31	4 424 345.63	944.4	1 637 568.79	888 383.74	58.1	6 940 996.35	2 895 682.48	373.4	437 270.36	186 849.60	40.8	867 138.55	Arkansas
California	12 999 075.03	5 647 148.17	533.7	5 434 044.02	2 906 132.58	218.6	11 361 648.66	5 714 588.44	321.1	14 898.29	8 789.99		2 816 646.82	California
Colorado	8 108 070.31	4 029 898.97	502.6	1 114 291.39	570 941.17	53.0	5 336 346.42	2 887 930.04	212.6	640 620.68	213 638.34	14.1	1 867 472.48	Colorado
Connecticut	3 062 872.02	1 259 558.60	73.6	199 024.56	98 423.00	4.9	3 262 054.48	1 045 804.04	54.1				967 409.36	Connecticut
Delaware	3 056 832.22	1 007 714.83	72.5	462 969.09	197 825.82	13.8	1 316 586.49	504 231.10	33.1				29 758.25	Delaware
Florida	961 134.07	461 470.92	48.8	1 415 010.74	681 331.64	25.4	8 673 290.35	4 193 570.44	246.6				950 514.00	Florida
Georgia	17 167 373.32	7 955 805.20	1 214.2	1 635 904.18	788 561.74	149.5	11 497 536.52	5 462 035.24	796.6	533 314.64	189 223.34	37.2	64 271.48	Georgia
Idaho	8 181 697.92	4 092 395.52	506.8	912 405.15	619 925.76	57.2	2 000 849.49	1 228 594.93	141.1	453 729.61	317 767.52	33.8	519 028.17	Idaho
Illinois	26 964 706.06	12 279 546.33	804.7	4 724 921.20	2 320 586.26	152.4	12 467 692.53	6 188 330.64	428.8				2 647 928.77	Illinois
Indiana	7 577 444.16	3 656 540.97	225.7	3 279 636.08	1 569 261.54	112.8	13 919 720.06	6 818 188.90	446.1				2 269 400.59	Indiana
Iowa	23 196 778.19	9 237 031.86	1 682.9	2 631 765.75	1 137 507.61	202.3	6 787 841.34	3 097 049.47	461.3	2 219 084.50	931 300.00	81.8	833 248.06	Iowa
Kansas	17 084 136.48	6 043 176.80	502.7	6 691 434.86	2 219 109.42	192.6	14 727 668.95	5 931 124.70	611.1	2 740 075.18	1 096 548.36	129.0	9 329.73	Kansas
Kentucky	10 822 980.31	4 613 947.28	429.4	2 201 360.16	869 563.46	88.8	8 405 230.67	3 830 529.40	309.3				1 057 698.86	Kentucky
Louisiana	8 488 463.18	3 636 143.36	661.2	1 524 997.08	745 374.93	127.7	4 794 026.93	2 388 493.13	284.5	118 433.78	59 216.89	0.6	436 213.69	Louisiana
Maine	6 911 058.78	3 299 935.38	230.7	976 338.68	469 161.81	39.0	1 031 111.02	490 979.81	35.9				829 895.00	Maine
Maryland	6 760 044.42	3 213 321.78	243.2	358 411.02	179 205.49	16.4	2 801 989.31	1 199 249.89	85.3	166 738.11	56 650.00	4.2	1 522.84	Maryland
Massachusetts	10 191 202.02	4 105 727.22	232.8	318 961.44	111 575.42	6.2	6 457 832.90	2 063 941.50	105.7	614 513.78	213 201.89	12.6	1 435 233.97	Massachusetts
Michigan	13 434 135.07	6 060 612.23	494.5	902 734.87	428 447.25	44.5	15 938 028.85	7 543 209.56	533.3	153 170.00	81 585.00	5.4	1 765 917.96	Michigan
Minnesota	24 037 561.24	9 885 843.07	2 292.0	6 378 124.65	2 852 798.97	429.2	6 304 972.38	2 526 600.00	665.1	679 082.36	46 300.00	67.8	6 876.96	Minnesota
Mississippi	7 888 193.89	3 828 941.39	655.0	1 466 853.97	694 789.48	99.0	8 592 005.48	4 289 227.97	513.6	441 631.06	220 765.52	37.4	497 648.64	Mississippi
Missouri	11 352 027.70	5 245 899.18	803.5	2 446 556.80	1 214 687.84	134.5	23 887 359.86	10 043 178.40	855.4	3 897 800.57	1 001 723.25	147.3	434 699.33	Missouri
Montana	8 867 279.16	4 384 335.12	791.4	1 128 667.67	804 270.67	110.9	2 038 184.14	1 505 217.92	180.6	1 038 200.22	590 707.66	117.2	3 581 886.63	Montana
Nebraska	7 876 337.16	3 714 691.59	1 440.4	422 885.75	198 859.17	27.6	8 295 840.26	4 061 116.80	886.5	476 496.06	238 248.01	55.6	3 238 030.43	Nebraska
Nevada	3 460 245.52	1 853 624.98	226.6	1 169 995.85	987 072.91	97.5	4 623 107.75	3 857 914.62	395.6				191 708.49	Nevada
New Hampshire	3 075 750.19	1 487 867.58	171.3	849 260.28	391 365.29	29.9	992 047.86	466 732.70	30.9	40 305.73	19 027.23	1.6	69 971.20	New Hampshire
New Jersey	7 623 795.12	2 561 531.49	148.7	1 907 419.04	489 910.00	28.7	8 714 399.91	2 622 731.24	61.1	281 862.31	85 955.00	5.7	729 109.27	New Jersey
New Mexico	5 306 286.45	2 758 849.68	714.3	1 479 309.93	974 985.81	125.6	6 066 726.70	3 953 589.25	626.2	250 983.39	154 354.77	16.7	747 652.49	New Mexico
New York	18 862 742.49	8 257 844.44	572.7	2 005 973.68	821 459.81	56.5	30 460 474.84	11 411 244.38	677.4	5 758 438.00	1 629 485.00	105.6	4 588 114.37	New York
North Carolina	12 567 732.97	5 676 757.66	884.7	5 397 099.84	1 931 114.21	150.6	7 195 885.05	2 907 428.14	200.8	1 142 317.45	522 193.02	34.3	1 256 757.97	North Carolina
North Dakota	9 088 973.11	4 418 505.42	1 587.9	1 416 495.81	691 160.31	287.9	2 836 909.40	1 424 063.85	384.8	362 221.75	181 110.83	33.4	1 648 815.59	North Dakota
Ohio	33 122 751.43	11 879 917.99	962.5	6 303 843.08	2 160 246.30	143.7	11 032 958.07	3 925 662.41	314.4	1 784 535.68	704 254.15	55.5	1 470 183.15	Ohio
Oklahoma	12 986 865.26	5 888 852.03	497.3	4 518 160.07	2 183 240.84	190.2	7 391 945.19	3 350 296.50	323.8	974 215.64	413 920.04	83.6	700 394.59	Oklahoma
Oregon	12 082 873.17	5 819 093.79	655.6	1 209 085.39	707 463.39	95.4	3 235 246.57	1 906 755.73	147.4	150 205.00	56 343.61	3.1	16 502.48	Oregon
Pennsylvania	36 825 248.98	14 114 694.79	729.7	2 356 274.24	910 692.90	49.7	22 570 675.04	6 018 252.50	370.8	2 568 397.38	777 205.00	51.5	2 780 770.81	Pennsylvania
Rhode Island	1 774 397.25	779 227.96	46.0	744 122.20	309 710.13	16.7	1 295 720.34	371 918.84	17.8				472 184.07	Rhode Island
South Carolina	9 016 476.73	4 124 045.22	924.4	1 632 597.63	752 239.06	240.5	6 059 138.86	2 312 102.40	420.7	260 738.96	64 170.33	21.7	434 989.00	South Carolina
South Dakota	8 674 597.86	4 244 636.27	989.8	2 472 927.36	1 298 070.59	362.5	6 597 432.10	3 124 925.22	865.6	220 684.20	4 000.00	45.5	47 047.92	South Dakota
Tennessee	6 805 683.36	3 313 936.07	269.6	4 992 648.46	2 466 250.77	170.5	12 111 693.38	5 497 891.43	432.9	530 314.24	265 158.61	36.9	481 400.12	Tennessee
Texas	42 341 998.56	16 190 624.91	3 122.8	6 230 119.86	2 485 625.11	399.5	24 153 852.77	9 770 989.33	1 450.3	6 087 369.89	2 264 879.36	359.8	1 012 094.29	Texas
Utah	3 304 423.76	1 895 805.92	219.0	1 379 283.77	875 315.41	130.5	4 128 493.86	2 708 675.15	242.3	216 416.46	141 858.70	23.6	494 817.82	Utah
Vermont	1 922 114.16	942 769.12	74.4	211 394.16	104 196.51	7.5	2 063 399.68	980 721.61	51.6	28 427.26	14 213.62	0.1	492 078.14	Vermont
Virginia	10 035 301.48	4 801 782.43	562.5	1 350 803.33	634 648.23	58.5	10 445 805.93	4 833 937.42	356.4	624 765.32	249 954.09	24.2	72 630.83	Virginia
Washington	11 384 615.67	6 290 895.45	457.0	1 133 589.22	492 402.07	61.8	3 300 889.79	1 566 300.00	130.6	403 543.60	187 500.00	13.3	349 480.48	Washington
West Virginia	6 489 747.95	2 365 041.63	255.6	1 555 196.35	716 620.21	55.6	4 473 244.28	1 846 014.09	143.2	77 099.55	27 448.00	0.4	700 008.17	West Virginia
Wisconsin	18 753 903.15	7 441 033.57	1 325.3	1 836 635.07	884 637.78	75.3	3 528 508.79	1 693 415.16	155.6	63 417.25	26 708.00	3.9	3 632 756.49	Wisconsin
Wyoming	6 127 625.61	3 078 098.70	687.6	1 718 492.72	1 056 194.97	192.0	3 696 057.10	2 297 128.40	220.8	86 301.49	55 793.00	8.4	200 135.93	Wyoming
Hawaii													366 625.00	Hawaii
TOTALS	549 655 391.27	237 852 399.82	32 452.9	101 601 856.04	47 416 540.83	6 524.6	391 163 750.23	173 503 011.88	17 609.1	36 282 346.59	13 744 109.45	1 765.5	52 608 838.02	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 121 406 643.87 Federal aid $ 53 678 414.10 Miles 4 850.4

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73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
74. State Highway Mileage and Expenditures for the Calendar Year 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

APPORTIONMENT OF FEDERAL AID FUNDS AUTHORIZED TO BE APPROPRIATED BY H. R. 4971 FOR THE FISCAL YEAR 1926

STATES	AMOUNT	STATES	AMOUNT
Alabama	$1,541,870	Nevada	$948,076
Arizona	1,056,171	New Hampshire	365,625
Arkansas	1,264,164	New Jersey	935,082
California	2,472,636	New Mexico	1,185,166
Colorado	1,373,237	New York	3,657,096
Connecticut	474,801	North Carolina	1,699,168
Delaware	365,625	North Dakota	1,180,699
Florida	892,878	Ohio	2,789,588
Georgia	1,983,089	Oklahoma	1,755,105
Idaho	936,927	Oregon	1,179,668
Illinois	3,191,479	Pennsylvania	3,360,123
Indiana	1,938,693	Rhode Island	365,625
Iowa	2,070,396	South Carolina	1,052,549
Kansas	2,074,360	South Dakota	1,215,020
Kentucky	1,411,607	Tennessee	1,622,985
Louisiana	997,262	Texas	4,415,715
Maine	685,140	Utah	846,467
Maryland	635,783	Vermont	365,625
Massachusetts	1,090,118	Virginia	1,449,713
Michigan	2,225,227	Washington	1,118,987
Minnesota	2,124,151	West Virginia	797,295
Mississippi	1,291,960	Wisconsin	1,873,308
Missouri	2,417,727	Wyoming	934,947
Montana	1,548,473	Hawaii	365,625
Nebraska	1,581,969	Total	73,125,000

Clemson University
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